RURAL WATER PIPELINE
FLOW AND WATER QUALITY STUDY

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ABSTRACT

Many farms and agricultural operations in rural Saskatchewan obtain water from rural water pipelines. These pipelines serve a small number of users spread over a large area, which results in large hydraulic residence times (HRT). The magnitude of the HRTs are a concern because of the degradation of water quality over time and the potential for biofilms to form in the pipeline. Biofilms have been reported to form even in the presence of a substantial chlorine residual, given time and substrate. The danger for users of these systems is that mature biofilms often have a diverse population of bacteria, some of which are reportedly opportunistic pathogens.

Despite the widespread use of rural water pipelines, little is known about the hydraulic patterns and water quality variations in these systems. This study investigated both hydraulic and water quality variations and sampled sections of the two pipelines studied for the presence of biofilms. Of the two pipelines studied, one conveyed raw water, while the other was supplied treated water from a regional water treatment plant.

The data collected over a period of approximately 16 months showed that average demand was highest in the winter months when many farmsteads were wintering cattle. However, peak hourly flow rates occurred in the late summer, typically August. The peak factors determined from the data showed a peak day factor of 1.97 and a peak hour factor of 4.53. Pressures in the extremities of the two networks studied were highly variable, sometimes reaching unacceptably low values during periods of high demand as a result of activity elsewhere in the network.

Peak concentrations of organic carbon and epifluorescent bacteria and high chlorine decay typically coincided with the peak temperatures, occurring in August and September of each year. The recommended threshold values for biofilm control (BDOC < 0.2mg/L, free chlorine > 0.1 mg/L, and temperature < 15 °C) were breached for only a portion of this period. The ground temperature, which varies throughout the year, is believed to greatly mitigate biofilm formation as water temperature in these branch

networks rarely exceeded the threshold value and few viable organisms were recovered from the pipe samples during biofilm analysis.

Hydraulic modelling was conducted to determine the HRT of the treated system and determine the chlorine decay coefficients (k). HRT values varied from 43 hours near the head of the branch network up to 241 hours at the Farpoint monitoring site. The model also showed another user (not part of the study) at an endpoint of the system may see HRTs as high as 585 hours.

Average values of chlorine decay coefficients determined in the study for the sites monitored varied from 0.3/d to 0.12/d for residence times of 2.2 days and 7.8 days, respectively. The highest individual decay coefficient value coincided with peak temperatures and DOC concentrations. This value was calculated to be 0.42/d for an HRT of 2.1 days. A mathematical relationship between DOC, temperature and chlorine decay coefficient could not be determined within the scope of this project.

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- Saskatchewan Water Corp. - Access to Facilities and Records
- Saskatchewan Research Council – Turbidimeters
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Thank you to my daughter Sophia. You have inspired me in ways you will never know. In two short years you have given me some of the happiest moments of my life

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1.0 INTRODUCTION

This chapter describes the project background, thesis objectives and the content of the chapters following.

1.1 Background

Rural Saskatchewan is home to many families and agricultural operations. Water in rural Saskatchewan is typically of poor quality, and available volumes are often inadequate with respect to quantity and reliability. These challenges illustrate the need for a safe, secure supply of water for rural residents. In response to the poor water quality and quantity available to rural individuals, government agencies such as the Prairie Farm Rehabilitation Administration (PFRA) and Saskatchewan Water Corporation (SWC), user groups such as the Saskatchewan Association of Rural Water Pipelines (SARWP) and other rural stakeholders have facilitated the construction of regional water pipelines which provide a higher quality, more reliable supply of water to rural users in many areas of Saskatchewan.

The distance between users in a rural setting may result in long hydraulic residence time (HRT) within a pipeline before water reaches the consumer. The potential for water quality deterioration and subsequent re-growth of bacteria within the pipeline is increased by long HRTs. This places these regional pipelines and distribution systems at risk for the development of biofilms. Biofilms have been known to harbour heterotrophic and pathogenic organisms, both of which can be released in high concentrations during periodic shedding of portions of the biofilm.

There is uncertainty surrounding the actual water quality and flow patterns in rural water pipelines. Much of this uncertainty stems from a lack of monitoring records and published formal studies. Therefore, several agencies combined resources to create and facilitate the current study.

1.1.1 Project Creation

This research program was created to monitor parameters related to water quality and biofilm development in rural water pipelines, as well as to assess the absence or presence of biofilms, and degree of water quality deterioration throughout the length of these lines. Water quality deterioration was anticipated by many to be significant. As part of an initial assessment of the potential for water quality deterioration in rural water pipelines, a literature review was conducted by Dr. Gordon Putz of the University of Saskatchewan (Putz 2000). Despite an exhaustive search, Dr. Putz was unable to find any articles specifically relating to water quality in rural water pipelines. Therefore, he reviewed the available literature on water quality deterioration, pertaining to urban distribution systems, believing that because the mechanisms of water quality deterioration are the same in either the rural or urban setting, much of the information and theory would be equally applicable. The degree of deterioration of water quality parameters affected by residence time would be expected to be amplified by the increased residence time in rural systems. The recommendations of the literature review by Dr. Putz provided the framework and focus of this study. Support for the research was provided by SARWP, PFRA, the University of Saskatchewan, and NSERC.

Two rural water pipelines were selected for detailed study. One delivered treated water, the other supplied raw water with point of use treatment by the users. Each were similar in physical infrastructure and were believed to be at risk for water quality deterioration and biofilm formation.

1.2 Project Objectives

The literature review completed by Dr. Putz (Putz 2000) contained several recommendations, which were adopted as the objectives for this thesis. Over time and with ongoing analysis, the original objectives were revised to focus on areas requiring more detailed study. These objectives are summarized below:

1. Hydraulic Monitoring: Characterize consumption rates, water use patterns and pressure fluctuations in rural water pipelines;

2. Water Quality Monitoring: Investigate and document changes in water quality throughout the length of the pipeline and from season to season by periodic collection of key water quality parameters. Key parameters include temperature, chlorine residual, dissolved organic carbon (DOC), biodegradable dissolved organic carbon (BDOC), as well as viable and total suspended heterotrophic bacteria counts. Compare the observed values to the published suggested threshold levels for control of biofilm growth;

3. Biofilm Monitoring: Document water quality parameters to determine if the potential for biofilm development exists and quantify attached bacterial biomass through sampling of the pipe surface; and

4. Hydraulic Residence Time Modelling: Collect physical and operating data from selected rural water distribution systems with the intent of creating a computer model of the distribution system to estimate residence times and variations thereof over the year. Identify potential problem areas in the pipeline. Determine chlorine decay, and investigate the relationship of the decay values with dissolved organic carbon levels and temperature based on average HRTs.

1.3 Scope

The scope of work includes the collection and interpretation of the above noted parameters for the two pipelines studied as well as modelling of the HRTs, calculation of the chlorine decay, and investigation of the relationship between decay, DOC and temperature for one of the pipelines. The study does not include the development of bacteriological models, DOC uptake models or chlorine decay modelling using the computer modelling software.

1.4 Contents

Chapter 1 describes the background to this study and the objectives of the work. Chapter 2 reviews the literature regarding characterization of flow in rural water pipelines, water quality issues, control of biofilms, effects of biofilm formation on distribution system water quality, efficacy of re-growth suppressants, and pipeline flow

and quality modeling. Chapter 3 describes the two study sites. Chapter 4 contains the sampling and analytical methods. Chapter 5 includes presentation, analysis and discussion of the hydraulic data recorded. Chapter 6 includes presentation, analysis and discussion of the water quality data and biofilm analysis. Chapter 7 includes presentation, analysis and discussion of the computer model input data, model output, and HRT estimations. Chapter 7 also presents and discusses the results of the chlorine decay analysis. Chapter 8 outlines the conclusions reached by this project and provides recommendations for future study and research.

2.0 LITERATURE REVIEW

This chapter discusses theories and observations from published works relevant to this study.

2.1 Purpose of Review

Hydraulic residence time (HRT) is characterized as the time it takes for a finite volume of water to be conveyed from the source of the water to the consumer. The physical configuration of rural distribution systems can lead to long HRT, which may result in deterioration of water quality to unsafe levels. The poor quality water resulting from long HRTs potentially contributes to the proliferation of organisms, some of which may be pathogenic, either in the bulk water or attached to the walls of the pipe itself. Bacterial colonies attached to the walls of distribution system pipes and substances created by their existence are termed biofilm.

The review provides the background information and terminology necessary to understand how the flow regime of rural distribution networks could promote water quality deterioration and/or biofilm growth.

Much of the available literature has focused on the deterioration of water quality as it pertains to the formation of biofilms in North American and European urban distribution networks. No publications relating to rural networks in climates similar to that in Saskatchewan could be located for this review.

2.2 Saskatchewan Rural Water Pipelines

2.2.1 Background

The province of Saskatchewan can be characterized as semi-arid and sparsely populated with the primary land use being agriculture. Although the Canadian Shield

covers nearly half of the province, the majority of residents live south of the Canadian shield, in the prairie region. Drinking water outside of urban centres comes from wells, rivers, creeks or shallow impoundments which store seasonal runoff. These supplies are often limited in quantity, of poor quality and unreliable (Pochylko et al., 1999). Rural residents in the prairies are often separated by significant distances, and are typically involved in intensive livestock operations, and cereal or vegetable crops or other agricultural activities.

2.2.2 Development

The Prairie Farm Rehabilitation Administration (PFRA) has been helping rural residents with the development and improvement of watersheds, wells, dams, and dugouts since 1935 (Pochylko et al., 1999). It is believed that the first rural water pipeline in Saskatchewan was developed by a group in the Lanigan area using polyvinyl chloride (PVC) pipe in 1967 (Pochylko and Morrison, 2000). The PFRA mandate was expanded in 1981 to include rural water pipelines when the Rural Water Development Program (RWDP) included rural water pipelines as projects eligible for technical and financial assistance (PFRA, 1985). High-density polyethylene (HDPE) is the most commonly used pipe material for rural water pipelines. It was first used in a Saskatchewan rural water pipeline at Tuxford in 1986 (Pochylko and Morrison, 2000).

Typically, a number of individuals interested in receiving water through a pipeline will form a group to distribute the capital and operating costs associated with the distribution system. Pochylko et al. (1999) note that these groups can consist of any combination of the following entities: individual agricultural producers, acreages, Hutterite Colonies, villages or hamlets, rural municipalities, or municipal districts.

Several rural networks have expanded off of regional pipelines owned and operated by the Saskatchewan Water Corporation (SWC) or by PFRA. These networks serve thousands of rural homes over areas of several hundreds of square kilometers. Pochylko and Morrison (2000) noted that there is estimated to be more than 2000 service connections in rural Saskatchewan supplied through over 3000 km of rural water pipeline.

2.2.3 Design and Construction

Rural and urban distribution systems are quite different. A brief description of each is presented in the following sections to illustrate the differences.

2.2.3.1 Urban Networks

Most modern urban networks typically avoid the use of dead ends and are looped systems constructed using PVC pipe. The design flow rate for the pipeline is usually the greater of the design fire flow (as identified in the Fire Underwriter's Survey, 1991) plus the average day user consumption, or the peak hourly user consumption. A minimum pipe diameter of 150 mm is recommended by Saskatchewan Environment (2002). Service connections to individual users are often flexible copper or polyethylene. The diameters of service connections typically range from 19 to 37 mm and are less than 20 m in length. The service connection is connected directly to the building plumbing, in most cases, and pressure for operation of fixtures is supplied from the distribution system pumps. For example, flow rates to individual users in Saskatoon are, on average, 2.5 Lpm (litres per minute) assuming a family of four and a per capita daily use of 225 litres (Putz, 2000). Urban water distribution networks are designed such that the water velocity at design flow ranges from 0.75 to 1.5 m/s. The high density of users typically leads to a hydraulic residence time in the system of only a few days at most.

2.2.3.2 Rural Networks

Pochylko and Morrison (2000) state that the major challenge in the design of rural water pipelines is economically providing water to clients who are, on average, approximately 1.5 km apart. Due to this distance and the low population density of users, a large diameter, high pressure network such as that typically present in urban centres is physically and economically undesirable.

Pochylko and Morrison (2000) identify the following approaches for design of rural water pipelines:

1. The system layout is based on a series of branches with no loops;
2. Fire flows are not considered as a system demand;

3. Each client receives intermittent delivery over a 24 hour period to satisfy average daily needs;
4. Each client of the pipeline installs a storage cistern complete with a pressure system to satisfy their short term peak demands and to ensure a supply of water during periods when the pipeline is not operating; and
5. The pressure at each client connection is not guaranteed.

Rural water pipelines are typically constructed of HDPE pipe ranging in size from 40 to 200 mm, the predominant diameters being 40 to 50 mm (Pochylko and Morrison, 2000). Individual service connections are also usually HDPE, 25 mm in diameter, running from the main supply line or branch line to the storage cistern (Putz, 2000) where it is discharged above the surface to avoid siphoning due to hydraulic transients in the main. Figure 2.1 shows a typical configuration.

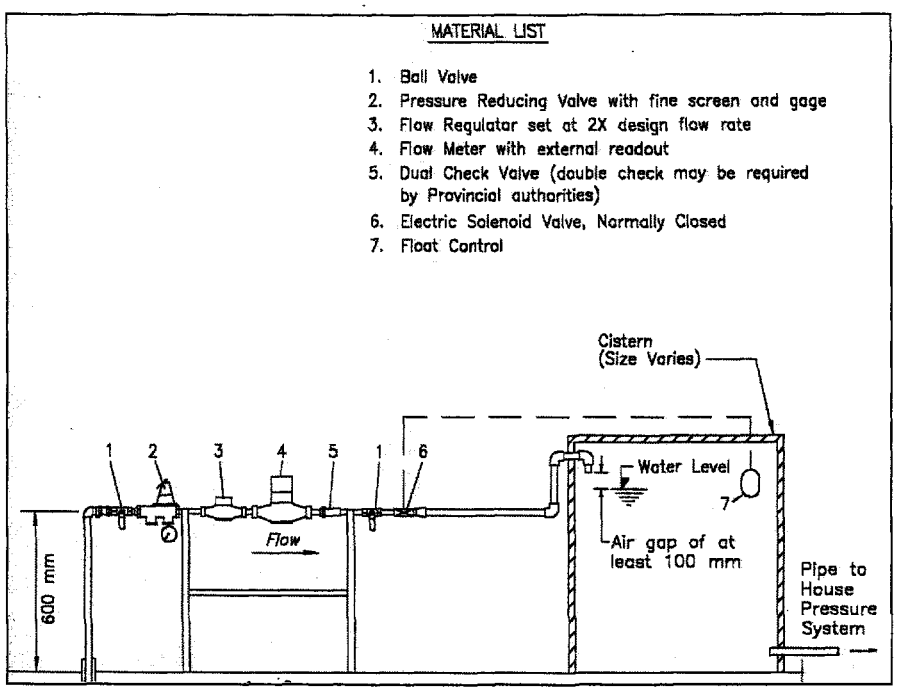

Figure 2.1 Typical arrangement of rural water service cistern, flow control and monitoring equipment (PFRA, 1999).

A solenoid and float system is typically employed on rural water pipelines for flow control into the cistern. As water is used, the level in the cistern drops to below the fill level and the float is tipped. This causes the solenoid valve to open, allowing water

from the line into the cistern. When the float is tipped at the full level, the solenoid closes, stopping the flow of water.

Flow rates are often limited through the use of flow regulation devices to 4.5 to 13.5 Lpm per service connection for a typical farm, and up to 118 Lpm for intensive livestock operations and small communities (Pochylko, 1999). Rural water distribution networks are designed such that flow velocity stays below 1 m/s. The low density of users usually leads to lengthy hydraulic residence times compared to urban networks.

2.2.4 Demand Characteristics

Few rural water utilities collect flow and pressure variations or seasonal consumption patterns. Data for the Avonlea and Swift Current pipelines are shown in Figures 2.2 and 2.3. Avonlea is located approximately 270 km southeast of Saskatoon, Saskatchewan (approximately 50 km southeast of Moose Jaw) and had 32 to 36 users over the period shown. Semi-annual volumes were reported. Swift Current is located at the junction of Highways #1 and #4, approximately 250 km southwest of Saskatoon. The number of users grew from 42 to 98 over the period shown. Quarterly volumes were available from this utility. In both cases, the average user demand was estimated by dividing the total system volume by the number of users present.

The Swift Current pipeline shows peak demand in the summer months while the Avonlea pipeline experiences peak demands in the winter. The latter pattern is believed to be the norm in rural water distribution networks. Average quarterly user demand volumes for the periods shown are 74 m^3 for the Avonlea users and 116 m^3 for the Swift Current users.

2.2.5 Hydraulic Residence Time (HRT)

Issues surrounding hydraulic residence time were not previously considered as part of the design philosophy of rural water pipelines. Pochylko et al. (1999) note:

"Little attention was given to water quality considerations when rural water pipeline development was first undertaken. In every case, the water that was delivered through the pipeline system was a significant improvement over the quality of water the client already had. With evolving public health regulations, growing awareness of the importance of water quality, and with increasing size and complexity of rural pipeline

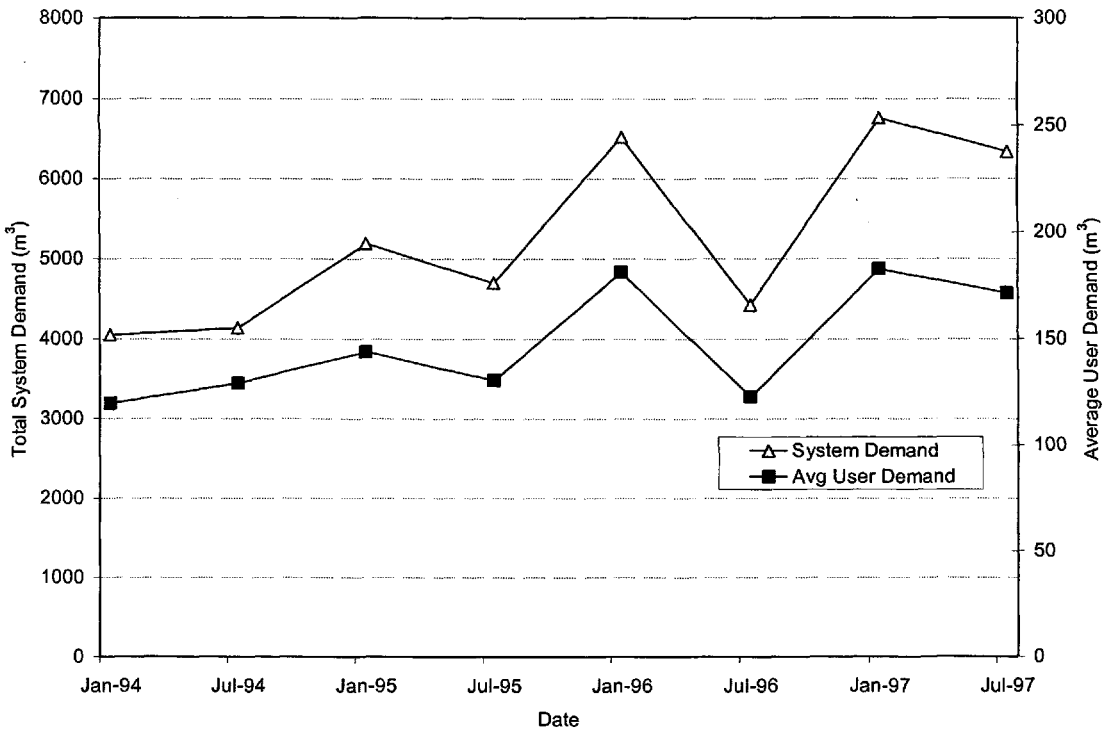

Figure 2.2 Avonlea pipeline semi-annual demand volumes (compiled from values received from PFRA).

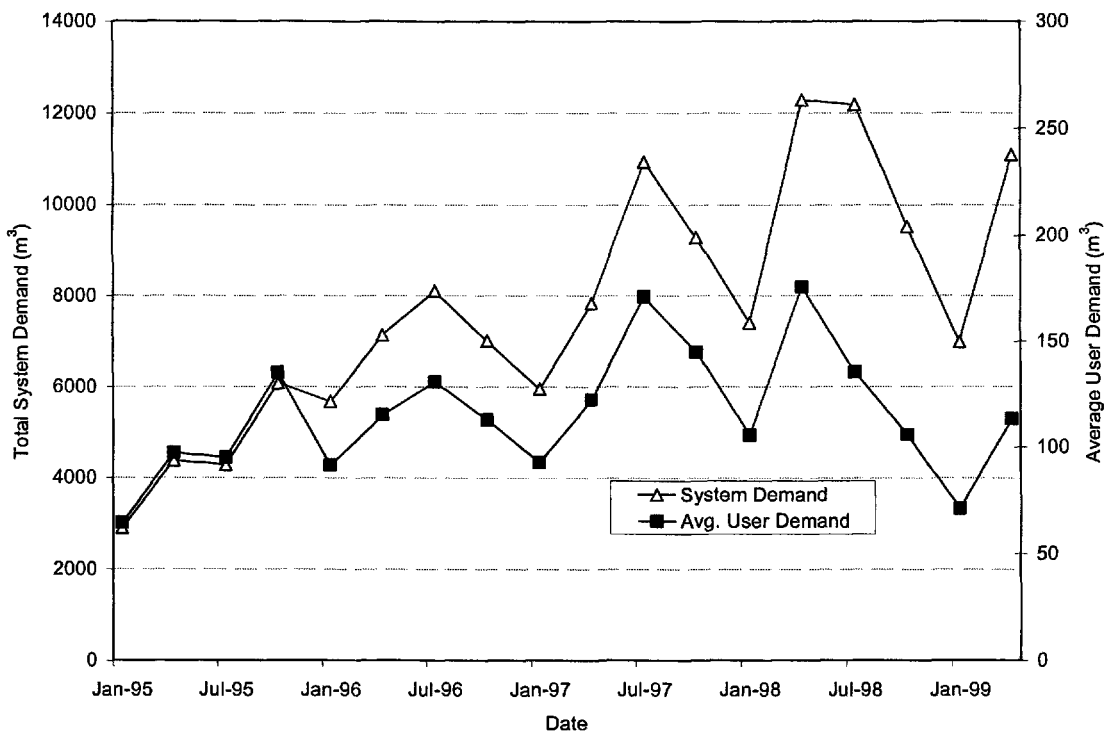

Figure 2.3 Swift Current pipeline quarterly demand volumes (compiled from values received from PFRA).

systems, the quality of water delivered through rural pipeline systems is becoming a greater concern."

The rural networks are created in a branch configuration. In some cases, a branch line has only one user at the end. To limit headloss over the length of the branch line, which can be up to 2500 metres, the line diameter is often larger than necessary for the volume being conveyed. This creates a large volume to flow rate ratio that increases residence time.

The level control (solenoid and float system) for the storage cistern in the user's home will generally cause flow in the service line to be intermittent. The time between solenoid actuations may be as much as several days. This allows ample time for reactions to occur within the bulk water in the service line and with material adhering to the interior of the service line. These reactions would still occur if flow was continuous, but as chlorine is consumed in reaction over a finite length of the service line, it is continuously replenished with a fresh parcel of higher quality water at the start of this finite length. Poor quality in flowing systems is likely limited closer to the terminus of the branches. Thus, the stagnant periods caused by cycling of water flow in rural branch lines may allow poor quality water to occur at the upstream end of these branches, resulting in more users along the service line experiencing poor water quality.

The concern over HRT stems from residual chlorine decay and bacterial growth being time-based functions. The question is: *Do conditions in rural water pipelines that cause the HRT to be long allow decay of the chlorine and growth of the bacteria before the water reaches the consumer?* If the water is consumed after decay of residual chlorine and growth of biofilm occurs, there is potential for illness due to the quality of the water.

2.3 Distribution System Water Quality

2.3.1 Factors Affecting Microbial Water Quality

The following sections discuss some of the issues surrounding distribution system water quality and the effect these issues have on bacterial re-growth. Prevost et al. (1998) stated that a distribution system is a very large bioreactor in which biofilms,

and micro and macroorganisms may thrive if adequate growth conditions are present. Van der Kooij et al. (1999) stated that pathogenic microorganisms in drinking water distribution systems might originate from re-growth and that opportunistic pathogens such as *Legionella* species (spp.), *Mycobacterium* spp., and *Pseudomonas* spp. can multiply in water systems and have been found to be responsible for waterborne diseases in many countries.

Niquette et al. (2001) summarized the principal factors influencing re-growth of heterotrophic bacteria (HPC) in distribution system pipes as:

1. The concentration of organic substrate in the treated water that could be utilized by heterotrophic bacteria;
2. The concentration of free residual chlorine in the distributed water;
3. The residence time of the drinking water in the distribution system from the treatment plant to the consumer tap;
4. The water temperature; and
5. The characteristics of the interior surface of the distribution pipes.

Figure 2.4 illustrates the parameters and interactions between the factors noted above.

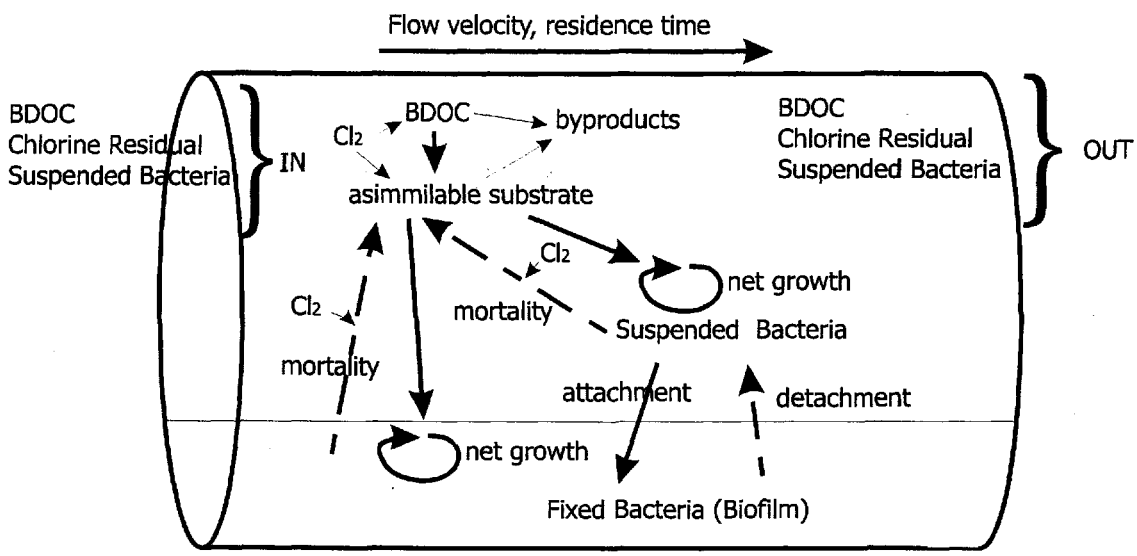

Figure 2.4 Interaction of factors affecting biofilm growth in distribution systems (adapted from Servais et al., 1995; Bois et al., 1997; Piriou et al., 1998).

The review presented herein discusses the above noted topics. In addition, attention is given to some of the mechanisms that affect the values of the above noted parameters.

2.3.2 Organic Carbon

2.3.2.1 Characterization

Organic carbon is present in most natural water sources and is made up of many fractions. Before the influence of organic carbon can be discussed the nature and relationships between these fractions should be characterized.

Natural organic matter (NOM) is a broad term used to describe a portion of the total organic carbon (TOC). NOM is a complex mixture of various carbohydrates and acids (Volk and LeChevallier, 2000), most of which is natural in origin and derived from living and decaying vegetation (LeChevallier, 1990). These compounds may include fulvic acids, polymeric carbohydrates, proteins and carboxylic acids (LeChevallier, 1990).

Dissolved organic carbon (DOC) is the fraction of TOC not removable by filtration, i.e. dissolved in the water. BDOC is that portion of dissolved organic carbon in water that can be mineralized by heterotrophic organisms (Huck, 1990). Volk and LeChevallier (2000) showed that BDOC is typically 5 to 21% of the DOC, on average, and is composed of 75% humic substances, 30% carbohydrates, and 4% amino acids. A portion of the carbohydrates and amino acids where reportedly bound with the humic substances. They note that BDOC can be characterized by three groups:

1. Monomeric substrates directly assimilable (utilized) by bacteria;
2. Polymeric organic matter that can be rapidly hydrolyzed (made soluble and thus consumable) by bacteria exoenzymes; and
3. Organic molecules that are slowly hydrolyzed by bacteria.

Assimilable organic carbon (AOC) is the portion of BDOC that can be converted into cell mass (Huck, 1990), or in other words, growth (Volk and LeChevallier, 2000). AOC typically makes up 0.1 to 9 % of the TOC (Van der Kooij, 1990), 2.4 to 44 % of

the DOC (Kaplan et al. 1993), and 0.2 to 30 % of BDOC (Volk and LeChevallier 2000). AOC is made up of low molecular weight constituents (LeChevallier, 1999; Escobar and Randall, 1999a), which typically include carbohydrates, carboxylic acids including formic and oxalic acid, amino acids, and aromatic acids (Van der Kooij et al., 1994). While BDOC is expressed in mg/L, the fraction of AOC is typically small enough that it is expressed in micrograms per litre (μg/L).

The intent of showing the organic fraction percentages determined by different investigators is not to debate their validity but to illustrate the variability in concentrations of BDOC and AOC as part of DOC or TOC in different waters. Some investigators (Volk and LeChevallier, 2000; Van der Kooij, 1992) developed mathematical relationships between DOC, BDOC, and AOC based on data from many water treatment plant effluents. The relationships were found to be statistically significant but generally had poor correlation coefficients, further illustrating the variability.

Concentrations of organic carbon fractions have also been noted to vary seasonally with peak concentrations occurring in summer or after periods of substantial rainfall (Volk and LeChevallier, 2000; Gatel et al., 1995; Liu et al., 2002; Escobar and Randall, 1999a; Niquette et al., 2001). Further discussion on the effects of temperature and rainfall is presented elsewhere in this review.

Volk and LeChevallier (2000) presented values for low, medium and high concentrations of organic matter based on data from the effluents of 95 North American water treatment plants. The results of their survey are shown in Table 2.1.

Table 2.1 Values of low, medium and high concentrations of organic matter in water. (Taken from Volk and LeChevallier, 2000).

Organic Fraction	Low	Medium	High
DOC (mg/L)	0.1	2-3	10-20
BDOC (mg/L)	> 0.1	0.3	1-1.5
AOC (μg/L)	0	50	>150

2.3.2.2 Utilization of Nutrients by Bacteria

Huck (1990) describes the term "re-growth" as follows:

> *"The term bacterial re-growth is commonly used to describe the phenomenon of bacterial growth in treated water, typically in the distribution system. Although it might be preferable to use the term bacterial growth, the terms re-growth and re-growth potential have become established in literature."*

Re-growth is defined differently in North America and Europe. In North America, re-growth is indicated by the presence of coliform species bacteria in distribution systems whereas in Europe, the term re-growth often refers to heterotrophic bacteria (a broader spectrum measure of microbial water quality) in the distributed water (LeChevallier, 1990). Use of the term re-growth in the context of this review refers to the European definition as coliforms are not always detectable when present.

Heterotrophic bacteria require carbon, nitrogen and phosphorus in the ratio of approximately 100:10:1 (C:N:P), with organic carbon often being the growth limiting nutrient (LeChevallier, 1990). Miettinen et al. (1997b) have suggested that phosphorus can be a limiting nutrient. Nitrogen sources have also been noted to promote growth of other types of bacteria (LeChevallier, 1990; Rittmann and Snoeyink, 1984).

AOC is believed by many investigators (Van der Kooij, 1992; Van der Kooij et al., 1994; Liu et al., 2002) to be the fraction of organic carbon most responsible for bacterial re-growth in distribution systems. Due to this relationship, these researchers also believe that AOC should be used as an indicator for re-growth potential. Some have used BDOC concentration alone as an indicator of re-growth potential (Servais et al., 1989; Prevost et al., 1998; Gatel et al., 1995). Others (LeChevallier et al., 1996b; Escobar and Randall, 1999a) note that degradation of BDOC or recalcitrant organics over time to AOC contribute to re-growth as well. These investigators have also expressed that in order for the researcher to realize the full re-growth potential, both AOC and BDOC concentrations should be considered.

Several researchers have investigated the limiting concentrations for controlling re-growth and coliform growth in distribution systems. These 'biostable' waters have insufficient nutrients to support biological activity. Van der Kooij (1992), who is

arguably the greatest proponent of using AOC as an indicator of re-growth, found that below an AOC concentration of 10 μg-C/L , heterotrophic plate counts (HPC) produced less than 100 colony forming units per millilitre (CFU/mL). The current Canadian guideline for HPC is 500 CFU/mL. LeChevallier et al. (1996b) determined that systems with AOC concentrations above 100 μg/L had 82% more coliform occurrences with coliform levels 19 times higher than systems with average AOC levels below 100 μg/L .

Nearly all reviewed publications using AOC concentration as an indicator of re-growth have shown a direct correlation between AOC consumption and HPC growth.

Gatel et al. (1995) studied BDOC levels as they correspond to the production of bacterial biomass in 41 supply systems operated by the Syndicat des Eaux d'Ile de France. The study showed that the allowable concentration of BDOC in the system was somewhat dependent on the concentration of the chlorine residual in the water. The report states that chlorine affects the ability of bacteria to use substrate. The BDOC could be consumed by the bacteria down to a threshold value below which no bacterial uptake was observed. This threshold value of BDOC was found to be higher with increased free chlorine concentrations.

None of the works reviewed were able to decisively find a correlation between bacterial growth and DOC or TOC and consequently no threshold values are known.

2.3.2.3 Measurement of Organic Carbon Fractions

Van der Kooij et al. (1994) noted that it is difficult to use chemical analyses to determine the concentration of all growth promoting compounds in water for the following reasons:

1. Concentrations of biodegradable compounds must be determined at very low concentrations;
2. A wide variety of compounds ranging from low molecular weight compounds to larger and more complex molecules, some of which contain biodegradable fractions, must be included in the analysis;
3. For many compounds, no data will be available about their significance as a source of energy and/or carbon; and

4. The importance of a number of compounds may depend on the nature of other compounds present (synergistic effects).

Van der Kooij et al. (1994) note that in light of these difficulties a method that incorporates a biological response is needed to asses the growth-promoting properties of drinking water.

AOC assays measure bacterial response to AOC. These assays are not a direct measure of the carbon concentration itself (Volk and LeChevallier, 2000). Van der Kooij et al. (1982) first reported a method of measuring AOC using known species of bacteria. *Pseudomonas flourescens* strain P17 was used as it commonly occurs naturally in drinking water, surface water and ground water. P17 (as it will be referred to herein) was reported by the authors to be able to use a variety of compounds at relatively high and low concentrations (a few micrograms per litre), does not need specific growth factors, and can use nitrate and ammonia as nitrogen sources. The downfall noted with using only P17 species was that the species did not utilize oxalic acid, which is commonly produced during ozonation of organics (Huck, 1990). A *spirillum* species, strain NOX (referred to herein as NOX) was later incorporated into the method as reported by Van der Kooij (1987) to consume the oxalic and formic acids not utilized by P17.

In the Van Der Kooij method, the species are subjected to a known concentration of carbon, usually an acetate solution, and the growth is monitored using HPC methods until a maximum population is reached. AOC of a water sample is calculated using the maximum population of these species observed in the water sample following inoculation and the yield values of the organisms on acetate. Other researchers have performed this assay monitoring for adenosine triphosphate (ATP) instead of HPC (LeChevallier, 1993). ATP is a critical organic compound utilized by bacteria to transfer energy during metabolism, the concentration of which is in proportion to the number of bacteria present.

Stanfield and Jago (1987) monitored ATP to determine the concentration of AOC in British waters. The method consisted of filtering indigenous bacteria from the sample and adding a known volume of inoculum. This inoculum was obtained from the

source water or distribution system from which the samples were taken (Huck, 1990). ATP was monitored until a maximum concentration was reached. The maximum ATP value was then converted to AOC concentration by means of a standard conversion factor.

A method of determining BDOC concentrations was proposed by Servais et al. (1987). This method involved filtration of the samples using a 0.2 μm filter, which was believed to remove the suspended particulate and protozoa that might affect the carbon concentration, without changing the portion that might be used by bacteria. The investigators felt that sterilization through heating had the potential to adversely modify portions of usable substrate. The 100 mL samples were then inoculated with 1 mL of filtered river water, which was believed to contain a wide variety of organisms. The authors believe this species diversity ensures that all portions of the available substrate will be utilized by one organism or another, reflecting the true amount of substrate in the sample. The method proposed by Servais et al. (1987) differs from the revised method proposed by Servais et al. (1989). The original method involved periodic sampling of the incubated waters to determine the biomass production and mortality rates of the bacteria, which provided valuable information on the kinetics of the substrate utilization, but was found to be labourious. The total mortality was divided by the growth yield to give an estimate of BDOC.

The second method proposed by Servais et al. (1989) was essentially the same as the first method as far as sample preparation and inoculation was concerned. However, rather than analyze the bacteria kinetics, the initial and final DOC concentrations were measured, the difference between them being the BDOC. Servais et al. (1989) reported good results using this method on several Belgian rivers. They noted very good agreement of this method using controlled doses of substrate in the range of 0.2 to 1.5 mg/L BDOC. The main criticism of this method is the length of the incubation period. This time far exceeds the residence time of any system and some believe that there would be development of bacteria in the latter stages that would not normally be present in a water supply system. Another potential pitfall can occur when neutralizing chlorine

residuals. Servais et al. (1987) reported an interference effect caused by sodium thiosulphate concentrations in excess of 20 mg/L .

With respect to the level of expertise required for conducting these analyses, the Servais (1989) method is far simpler to conduct. It does not require that a population of bacteria be sustained between tests and determine the kinetics or use biological enumeration techniques like the method of Van der Kooij (1987) or its variations. The method proposed by Servais et al. (1989) could be conducted at the water treatment plant (with the exception of the DOC analysis, which requires an expensive laboratory analyzer), with the purchase of a few items of glassware, filters and a vacuum pump. The initial and final samples could be sent to a laboratory for DOC analysis.

2.3.2.4 Effects of Water Treatment

2.3.2.4.1 Removal by Processes

The difficulty with organic carbon fractions lies in the fact that they are used as (or can be a source of) substrate for suspended and attached bacterial species and are not easily removed by treatment processes. Table 2.2 shows the documented removal of organic fractions for different types of processes.

DOC is composed of large, relatively high molecular weight particles and concentrations can be decreased in certain types of water treatment processes (refer to Table 2.2). Enhanced coagulation, GAC filtration and membranes have demonstrated some removal of BDOC but a residual concentration often remains. Biological filtration has been used by many to reduce organic compounds in drinking water processes. The constituents of AOC are typically low molecular weight, making removal through conventional water treatment or some types of nanofiltration membranes difficult (LeChevallier, 1999; Escobar and Randall, 1999a). Success in removal of AOC using these processes is largely attributed to the AOC particles being bound to the humic material or larger organic compounds targeted by the processes (LeChevallier, 1999).

In the case of membranes, the characteristics of the membrane, particularly pore size, in relationship to the size of AOC will greatly affect the ability of the membrane to

reduce these concentrations. Biological filtration has been shown to dramatically reduce AOC levels in finished water (Zhang and Huck, 1996).

Table 2.2 Comparison of organic matter reduction for common water treatment processes.

Process	% DOC reduction	% BDOC reduction	% AOC reduction
Nanofiltration Membrane	97 [1]	97 [1] – 100 [2]	n.c.
Ultrafiltration Membrane	68 [2]	n.c.	n.c.
Lime Softening	23 [1]	44 [1]	n.c.
Coagulation	25 [2]	30 [2]	n.c.
Enhanced Coagulation	43 [2]	38 [2]	n.c.
Biological Filtration	n.c.	71 [3]	65 [4] – 90 [5]

Sources: (1) Escobar and Randall (1999a)
(2) LeChevallier (1999)
(3) Prevost et al. (1997)
(4) Liu et al. (2002)
(5) LeChevallier (1990)
n.c. – Not considered.

Escobar and Randall (1999a) noted a marginal increase in AOC through a nanofiltration train, which they attributed to carbon impurities in the chemical feed (H_2SO_4 and proprietary anti-scalant). Despite the increase in AOC, Escobar and Randall (1999a) believed that the removal of DOC and BDOC by the membranes offered a significant advantage for maintaining distribution system water quality. The reduction of DOC and BDOC greatly decreases the amount of organic material available to be reduced to more assimilable compounds by the residual chlorine maintained in the distribution system.

2.3.2.4.2 Reaction with Oxidants

Oxidants, when in contact with organic material, have the ability to break up long chain organics into smaller more easily assimilable compounds (LeChevallier, 1990; Miettinen et al., 1997a; Liu et al., 2002).

Ozone has been shown by many to increase the concentrations of AOC (Van der Kooij et al., 1994; LeChevallier et al., 1996a; Escobar and Randall, 1999b), and BDOC (Escobar and Randall, 1999b) in process waters. Escobar and Randall (1999b) studied the effect of ozonation on the concentration of AOC and reported increases as much as 112 %. Carboxylic acids, which include oxalic acid used as substrate for NOX, have been documented in ozonated samples to make up as much as 90% of the 240 μg/L AOC present (Van der Kooij et al., 1994). The raw water AOC concentration for the same study was only 135 μg/L , 95% of which was useable by P17, which suggests not only an increase in the concentration of AOC but a degradation of the molecules to a more assimilable form. Ozone was also noted to increase BDOC concentrations by Escobar and Randall (1999b) by approximately 44%, on average. LeChevallier et al. (1996a) studied a number of water treatment plant effluents which showed that many of the facilities using ozone as a pre-oxidant, without biological filtration, had higher concentrations of AOC and BDOC in their effluents than others in the study. Raw water organic carbon fractions were not presented as part of that study so it is not known for certain whether the levels increased or decreased through the process at the individual sites.

Chlorine has also been noted to increase biodegradable fractions of carbon in drinking water, although typically to a lesser extent than ozone (Huck, 1990). Data contained in LeChevallier et al. (1996a) shows a higher level of biodegradable organics in water treatment plant effluents at facilities that used chlorine as a pre-oxidant than others who did not. Liu et al. (2002) showed increases in AOC concentrations from 20 to 240% as a result of post disinfection with chlorine. LeChevallier (1999) also reported 240% increase in AOC concentration at one water treatment plant in New Jersey as a result of pre-oxidation.

2.3.3 Oxidant Residuals in Distribution Systems

2.3.3.1 Reasons for Maintaining an Oxidant Residual

According to Van der Kooij et al. (1999), water providers face several challenges when supplying water to consumers. The first three of these points, speak directly to the maintenance of an oxidant residual:

1. Ensuring the microbiological safety of drinking water;
2. Minimizing concentrations of toxic by-products;
3. Achieving aesthetic quality; and
4. Providing consumers with drinking water at an acceptable price.

Some European countries, particularly the Netherlands, focus their drinking water treatment processes on the reduction of substrate. The rationale is that there will be no re-growth if there is no substrate available to organisms. In these cases no residual oxidant is required to inhibit re-growth (Van der Kooij at al., 1994). In North America, utilities are required by legislation to maintain a minimum concentration of oxidant at any point in the distribution system, regardless of the level of treatment employed. Saskatchewan Environment (2002) requires that a minimum free chlorine concentration of 0.1 mg/L be maintained everywhere in the distribution system.

North Americans populate a vast area and are subject to varied raw water quality. In many cases in Canada, particularly Saskatchewan, small towns and villages do not have the technological and financial resources required to implement the types of water treatment processes required for reducing available substrate. Consequently, to provide some assurance of safety and a measure of protection against re-growth, an oxidant residual is maintained, despite the fact that some consumers dislike the taste and smell of the residual in their drinking water. In Saskatchewan, free chlorine is commonly used to provide an oxidant residual. This review will focus on free chlorine as a residual oxidant.

Investigators have noted the following reasons for maintaining an oxidant residual:

1. Inactivation of organisms entering the distribution system which have escaped the effect of treatment (Van der Kooij et al.,1999; Trussell, 1999);
2. Limiting re-growth in the distribution system (Trussell, 1999; Van der Kooij et al.,1999; Haas, 1999; LeChevallier, 1999; Gatel et al., 1998);
3. Inactivate organisms that may enter the distribution system through contamination (Haas, 1999; LeChevallier, 1999); and
4. Provide an indication that contamination has occurred (Trussell, 1999; Haas, 1999).

Before these points can be discussed, a review of chlorine chemistry and the mechanism of bacterial inactivation is required.

2.3.3.2 Chlorine Chemistry

The following discussion relies heavily on Connell (1996). A more complete review of chlorine and chloramine reaction, usage, and dosage is provided by Connell (1996).

Chlorine participates in three reactions within the context of water treatment. Oxidation is the reaction where an exchange of electrons take place such that chlorine gains electrons from the chemical species being oxidized. Oxidation by chlorine is typically characterized in water treatment by the reduction of inorganic molecules such as iron and manganese present in many water supplies; or reduction of organic species. Substitution is the replacement of a portion of a chemical molecule with the chloride ion. The substitution reaction between ammonia and chlorine results in the generation of chloramines. The last reaction, which is most applicable to this review, is disinfection. Disinfection is characterized as the inactivation of living organisms in water.

Sodium hypochlorite (NaOCl) is commonly used in Saskatchewan for disinfection of water supplies. When added to water, NaOCl dissociates in the water and then forms hypochlorous acid (HOCl) and sodium hydroxide (NaOH) as shown in [2.1], [2.2] and [2.3].

[2.1] $NaOCl + H_20 \Rightarrow Na^+ + OCl^- + H^+ + OH^-$

[2.2] $H^+ + OCl^- \Leftrightarrow HOCl$

[2.3] $Na^+ + OH^- \Leftrightarrow NaOH$

HOCl is a weak acid and as such does not completely dissociate in water. Hypochlorous acid and hypochlorite ion concentration vary with pH. The percentage of each species for variations in pH are shown in Figure 2.5.

Bacteria in water have a slime layer coating the cell wall believed to facilitate the uptake of substrate (Christensen and Characklis, 1990). This layer is negatively charged. Hypochlorous acid, which has a neutral charge, is able to penetrate the slime layer and cell wall better than the negatively charged hypochlorite ion. Once inside, the HOCl interferes with metabolic and reproductive processes by altering enzymes and causing death or inability to reproduce, rendering the organism inactive. At lower pH, more HOCl is present, which is why greater inactivation can be achieved with pH values less than 7.5. HOCl is said to be more effective in penetrating the cell wall because it is a smaller, less complex molecule than, for example, monochloramine (NH_2Cl); and meets with less resistance due to its valence than the hypochlorite ion (Connell 1996).

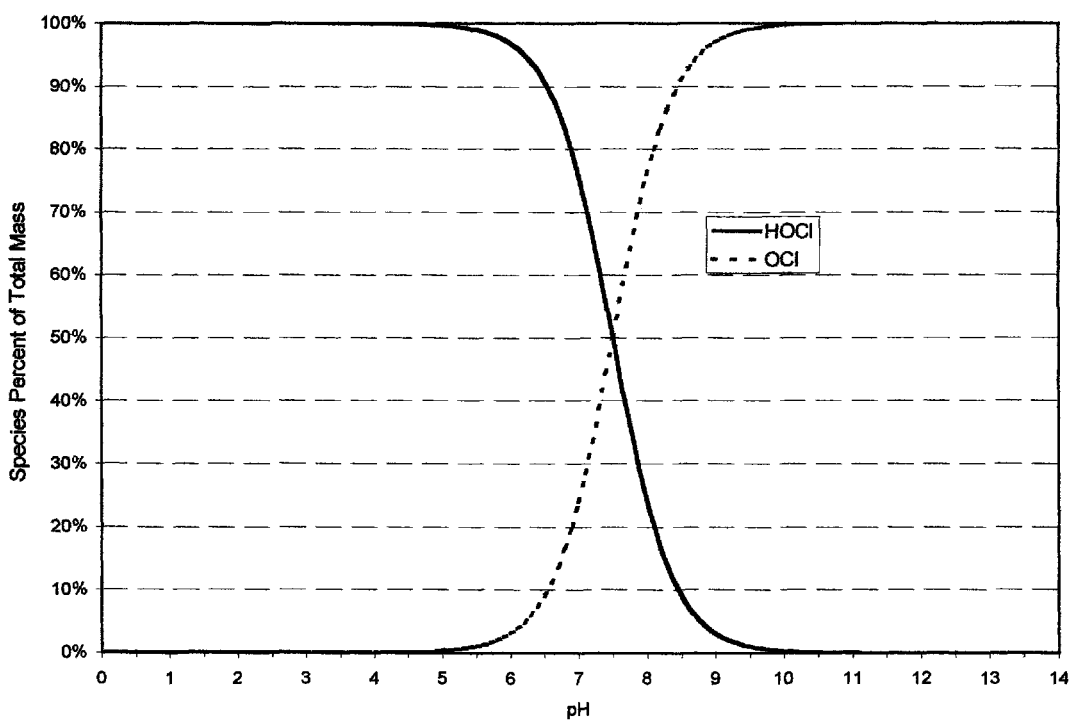

Figure 2.5 Ionization fraction diagram for HOCl and OCl^- for 25°C.

This is not to say that the hypochlorite ion will not penetrate the cell, but the speed of reaction is less due to the charge of the slime layer, requiring more contact time in order to penetrate.

2.3.3.3 Levels for Bacterial Control

Some re-growth is unavoidable. Consequently, the goal of the water supplier should be to control the extent of re-growth, not eliminate it (LeChevallier, 1999). Bacteria will be present in the water if substrate is available. Maintaining an oxidant residual often only limits growth to below acceptable levels of exposure. Table 2.3 shows reported values of chlorine residuals and their effect on HPC bacteria.

Table 2.3 Published chlorine residuals for maintaining bacterial control.

Chlorine Residual	Result	Citation
0.8 mg/L	HPC < 100 CFU/mL	Haas (1999)
1.2 mg/L	HPC < 100 CFU/mL	LeChevallier et al. (1987)
0.2 mg/L	HPC < 500 CFU/mL	McCabe et al. (1970)
0.2 mg/L	HPC < 100 CFU/mL	Piriou et al. (1998)

In their experimental loop, Gatel et al. (1998) noted that *E. coli* could be detected in the water column. As soon as chlorine was added to create a residual of 0.5 mg/L , the suspended coliforms in the water column were completely inactivated.

Not all bacteria are inactivated by chlorine. Some are merely injured by the effects of chlorine. LeChevallier et al. (1987) reported that most researchers have found coliform injury in drinking water to range from 60 to 99% of the total viable coliform population. Waters et al. (1989) investigated injured coliform bacteria and found that the coliforms could repair the cellular lesion and resuscitate in biofilms.

2.3.3.4 Factors Affecting Chlorine Decay

Chlorine decay in the bulk water phase is widely modelled as a first order decay reaction (Rossman, 2000; Haestad Methods et al., 2003).

[2.4] $M_i = M_o e^{-kt}$

where: M_i *is the mass of chlorine reaching the user at time 'i';*
M_o *is the mass of chlorine entering the system at time 0;*
t is the HRT (in days); and
k is the decay coefficient.

Knowing the initial and final chlorine concentrations over a time period, the decay coefficient can be calculated using [2.4]. Observed ranges of chlorine decay coefficient are presented in Table 2.4. It should be noted that the values presented below were determined by the authors over a period of 2 to 12 hours and were expressed as a daily decay coefficient for the purposes of presentation. Decay rates are often observed to decrease with increasing residence time as the easily oxidizable constituents react quicker than the more complex molecules. If observed over a period of several days, one could expect the average decay coefficient to be lower as decay rates decrease with time.

Table 2.4 Published ranges of chlorine decay coefficient (k).

k (d^{-1})	Source of Data	Citation
0.1 – 17.7	Distribution System	Vasconcelos et al.(1997)
2.64 – 6.0	Experimental System	Keine et al. (1998)
0.5 – 1.5	Distribution System	Prevost et al. (1998)
0.6 – 0.72	Storage Reservoir Observations	Gagnon et al. (1998)

A wide variety of factors can affect the amount and rate of chlorine decay in distribution systems, some of which are presented below:

1. Inactivation of suspended bacteria;

2. Oxidation of inorganic and organic compounds suspended in the bulk water, or as part of, or attached to, the pipe wall;
3. Temperature;
4. Hydraulic retention time;
5. pH; and
6. Inactivation of attached bacteria and reaction with extracellular polymeric substances (EPS).

Kiene et al. (1998) studied the relative importance of the factors responsible for chlorine decay in drinking water systems. Table 2.5 shows the relative contributions of bulk water, pipe material, pipe deposits, and fixed biomass to the decay of chlorine in a plastic pipe and a cast iron pipe, both 250 mm in diameter. In the two-hour test, the residual free chlorine concentration for the plastic pipe sample decreased by 0.22 mg/L. The change in free chlorine residual for the cast iron pipe sample was 0.5 mg/L .

Table 2.5 Relative consumption of residual chlorine by bulk water, pipe material, pipe deposits, and fixed biomass. (Taken from Kiene et al., 1998).

Material Chlorine Reacts with	Plastic Pipe	Cast Iron Pipe
Suspended and Dissolved Material in Bulk Water	26 %	11%
Pipe Material	2%	56%
Pipe Deposits	56%	25%
Biofilm Biomass	16%	7%

Lu et al. (1999) reported that where attached biomass i.e. biofilm (bacteria colonized on the surface of the pipe) is present, the bulk water appears to consume most of the chlorine unless the pipe diameter is relatively small ($\leq$ 40 mm). The surface to volume ratio (S/V) increases with decreased pipe diameter and the relative contribution of attached biomass to chlorine decay is increased. This is of more concern where high substrate concentrations have been available and the biofilm is well established. This

relationship could have great significance for rural water pipelines. Keine et al. (1998) established an empirical relationship between BDOC, surface area to volume ratio and decay rate caused by fixed biomass from experimental data.

Kiene et al. (1998) also studied the influence of pipe material on chlorine decay. They found that the plastic had a negligible effect on the decay of chlorine. Kiene and Levi (1996) have reported that in cast iron pipes, material corrosion and deposits are the principal consumers of chlorine. LeChevallier (1990) stated that corrosion inhibitors can improve the disinfection efficiency of free chlorine for biofilms on iron pipes. The oxidation of 1 mg/L of suspended iron requires 0.6 mg/L of chlorine, thus an immersed iron pipe may be responsible for large scale consumption of free chlorine as the pipe material is oxidized by the free chlorine. Oxidation of organic compounds by chlorine is also of growing concern since the discovery of trihalomethanes (THMs) in the 1970s (Trussell, 1999). In addition to the depletion of chlorine residuals through conversion of organic matter to more assimilable forms, reaction of chlorine with humic and fulvic acids creates disinfection by products (DBPs) some of which are known carcinogens.

HRT affects the residual as increased HRT allows for more reaction time between chlorine and the compounds present. Gatel et al. (1998) reported flushing to reduce HRT in the distribution system increased chlorine residual at the flushing location and reduced THM concentrations. Considering the state of water supplies in Saskatchewan and the economics of treated water production, water conservation is required, and flushing would only represent a short-term solution to residence time issues.

An increase in temperature often speeds the rate of reaction between chlorine and the compound chlorine is reacting with (Kiene et al, 1998). More rapid decay of the residual can be expected at higher temperatures.

2.3.4 Particulate Matter

It is known that bacteria have an affinity for attachment to solid surfaces. Biological filtration exploits this tendency by encouraging growth of bacteria on sand, expanded clay, or granular activated carbon. Given this affinity, particulate matter

carried into a distribution system may have bacteria attached. LeChevallier (1990) stated that the accumulation of sediment and debris in distribution system lines can provide habitats for microbial growth and protection from disinfection. Ridgway and Olson (1981) used a scanning electron microscope to examine the surface of particles and reported that 17% of the 10 to 50 micron diameter particles were colonized.

Particulate matter may also contain organic fractions, which can interfere with the maintenance of a chlorine residual (LeChevallier et al., 1981; LeChevallier et al., 1996b).

2.3.5 Rainfall

Rainfall can adversely affect the raw water supply for drinking water systems and treated water quality as stated by LeChevallier et al. (1996a).

> *"Rainfall can be a mechanism that introduces coliform bacteria into the system through leaks and cross connections. Rainfall can wash dissolved nutrients into the watershed and increase organic carbon levels. In times of drought, the lack of rain can result in proportionally higher levels of dissolved mineral or nutrients in the source water that can stimulate increased corrosion or bacterial (re-)growth in the pipe network."*

LeChevallier et al. (1996) observed a 38% increase in coliform occurrence within free chlorinated systems when rainfall increased from 13 to 76 mm. Rainfall may also wash nutrients into the raw water supply (LeChevallier et al., 1990b). LeChevallier et al. (1990b) observed a seven day lag between rainfall events and the occurrence of coliform bacteria in distribution system water samples. The lag observed was believed to be a period of transport and the lag growth phase of bacterial development after which higher bacterial densities were detected.

2.3.6 Temperature

Many of the rate dependent processes in water distribution systems are affected by change in temperature. LeChevallier (1990) stated that temperature affects treatment plant efficiency, microbial growth rate, disinfection efficiency and dissipation of oxidant residuals, corrosion rates, and distribution system hydraulics as well as water velocity through increased consumer demand. Bacterial activity and chlorine reaction rates are

two of the more important processes affected by temperature that will be presented herein.

2.3.6.1 Bacterial Activity as a Result of Temperature

Bacteria are more active at higher temperatures. The rate of cell division can increase dramatically for elevated temperatures. Some researchers have shown a statistically significant increase in bacterial numbers when the water temperature is above 15°C (Gatel et al., 1995; LeChevallier et al., 1996; Niquette et al., 2001), which suggests a threshold value may exist. Servais et al. (1995) presented a relationship between bacterial activity and temperatures based on experimental results of growth rates of bacteria in biofilters. This non-linear relationship, shown in Figure 2.6, is a plot of bacterial activity versus temperature, normalized to 20°C.

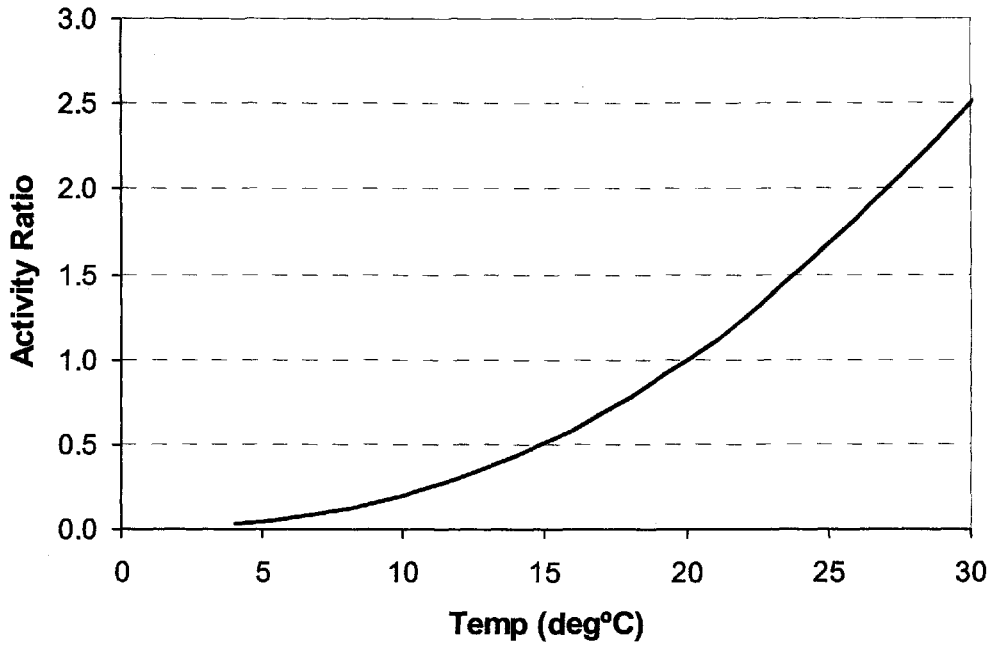

Figure 2.6 Bacterial activity, normalized to 20°C (adapted from Servais et al., 1995).

Temperature varies according to season, the effects of which have been reported by Neden et al. (1992) and Carter et al. (1998). Neden et al. (1992) reported a seasonal cycle for the occurrence of HPCs in which it was noted that the peak occurrences (HPC > 500 CFU/mL and coliform occurrence) were observed in August and September when

water temperatures were highest. In contrast, the lowest occurrences were observed in January and February. Carter et al. (1998) reported a similar seasonal variation.

2.3.6.2 Temperature and Chlorine

The influence of temperature on chlorine residual is directly related to the rate reaction of chlorine with the compounds it neutralizes. Kiene et al. (1998) developed a relationship between temperature, TOC, and chlorine decay coefficient based on the study of 21 water samples. This relationship is presented in [2.5] below.

[2.5] $k'_e = a \times [TOC] \times e^{(-b/T)}$

where: k'_e is the chlorine decay coefficient;
a, b are constants dependent on the characteristics of the organic matter;
TOC is the total organic carbon in the sample in mg/l; and
T is the temperature in degrees Kelvin.

The above relationship shows an increase in decay rate with an increase in temperature, TOC or both. Note that peak concentrations of organic matter typically coincide with peak temperatures (Niquette et al., 2001).

There is an concurrent effect as well. The increased bacterial growth present at elevated temperature or high organic matter concentrations and the inactivation of bacteria by chlorine places an increased demand on the chlorine residual.

2.3.6.3 Seasonal Temperature Variation in Distribution Systems

Prevost et al. (1998) compared portions of a Laval Quebec distribution system fed by biologically treated (St. Rose) and conventionally treated (Pont Viau) waters containing low levels of substrate and residual chlorine and higher levels of substrate and residual chlorine, respectively. The conclusion reached in the study was that in warmer temperatures, limitation of available substrate helps to inhibit re-growth but in cooler waters the presence of a chlorine residual may be enough to maintain the quality of the water.

Liu et al. (2002) observed several variations in a study of distribution systems in China. During the winter, when water temperatures were approximately 5°C, the investigators noted an increase in the AOC concentration throughout the length of the

pipeline. Consumption of AOC by bacteria was believed to be very low due to the temperatures and the increase in AOC was attributed to reaction of the chlorine with organic matter. An additional contribution to the increase in AOC may have been the cellular decay of inactivated bacteria, but this was not the focus of the study and consequently was not noted in their analysis.

The same study showed that in spring or autumn, the water temperature was more than 10°C. Increased bacterial activity due to temperature and accelerated reaction of chlorine was reported to be the cause of decreasing AOC concentrations throughout the distribution system. The HRT of the system during these seasons (approximately 5 hours) was credited with allowing complete reaction of chlorine with the organics in the water treatment plant clearwell before going into the distribution system (and prior to the sampling point). The resulting observation was that there was a constant decrease in AOC concentration.

Liu et al. (2002) noted that during the summer months, which had high water temperatures (> 20°C) and low HRT (approximately 2 hours), the AOC concentration was observed to increase first then decrease. Liu et al. (2002) explained this phenomenon as continued oxidation of organic material from the clear well into the first part of the pipeline, causing an increase in AOC concentration, followed by consumption of the AOC by distribution system bacteria throughout the remainder of the pipeline.

Liu et al. (2002) also noted that the systems that used groundwater as a raw water source had less variability in the consumption of AOC, which was attributed to the consistency of the groundwater temperature. As the groundwater temperature was higher than that in the surface water supplied lines in winter, a constant decrease in AOC concentration, indicating consumption by distribution system bacteria was reported for all seasons.

These studies illustrate the importance of the interactions between water temperature, chlorine residual, substrate concentration, and bacterial re-growth in distribution systems, and points to the magnitude of the effect of temperature on factors affecting biological quality.

2.3.7 Distribution System Effects

2.3.7.1 Operational Considerations

Contamination of distribution systems, which can contribute to re-growth through seed bacteria and depletion of chlorine residual, sometimes occurs as a function of operation or maintenance of the system.

Flushing programs have been shown to improve the water quality by removing sediments which can provide substrate for organisms and shelter from the effects of chlorine as well as increase chlorine decay (Ridgway and Olson, 1981; LeChevallier et al., 1981; LeChevallier, 1990; LeChevallier et al. 1996b) and decreasing HRT (Gatel et al., 1998).

Hydraulic transients can also adversely affect water quality by drawing contaminated water into the system through joints that would normally be sealed under high positive pressures. LeChevallier (1999) described on going research using a single pipeline model in which a 60-second hydraulic transient episode could result in 261 litres of potentially contaminated water being drawn into the pipeline during a sudden pump stop due to power failure. Sudden reversal of flows due to water hammer or sudden changes in velocity have also been suspected of shearing biofilms and releasing bacterial biomass into the water (LeChevallier, 1990; Opheim et al., 1988).

Breaks in the distribution main can and do occur. Typically the response of the utility is to isolate, repair, superchlorinate, and flush the affected section. LeChevallier (1999) warns that flushing velocities do not always remove contaminated debris and microbial testing using standard techniques may not identify injured coliforms, which can recuperate and present a problem within the distribution system at a later date.

Contamination can potentially come from cross-connections with non-potable systems or as a result of back-siphonage from uncontrolled water systems, such as domestic plumbing or industrial water supplies. Installation of backflow-prevention devices are a method of defense against this mode of contamination but routine maintenance and inspection to ensure proper function of these devices would be a challenge (LeChevallier, 1999).

Another contributor to deteriorating water quality can often be the age of the distribution mains. This is particularly true for distribution systems containing cast iron mains, which, if corrosion inhibitors are improperly or not used, have been observed to develop substantial tubercles from pipe corrosion. Nagy and Olson (1985) noted an increase in the number of HPC bacteria with material age. Nagy and Olson (1985) estimated that the increase in HPC concentrations is 1 log for every ten years a pipe is in service.

Van der Kooij et al. (1999) identify several measures for preservation of distribution system water quality employed in the Netherlands. The measures most applicable to the rural water distribution networks are listed below:

1. Maintaining a constant pressure (at any point) in the distribution system of approximately 200 kPa (29 psi);
2. Installation of valves in all residential service connections to prevent back-siphonage;
3. Installation of tanks in connections to facilities which may potentially contaminate the system that are filled by a line discharging above the tank water surface (i.e. air gap); and
4. Development of an optimized scheme for microbiological water quality monitoring.

2.3.7.2 Dead End Pipes

Rural distribution systems are comprised of dead end small diameter pipes, which puts them at risk for the issues discussed herein. Several authors have investigated the effect of stagnation in dead end pipes (Prevost et al., 1997; Carter et al., 1998).

Prevost et al. (1997) conducted tests on three distribution systems, each employing different treatment processes, at several locations of varying HRT. After an overnight stagnant period, the following general observations were made as the lines were flushed:

1. Bacteria consistently decreased;

2. Chlorine residual consistently increased;
3. BDOC concentrations usually increased; and
4. Temperature decreased.

Prevost et al. (1997) indicated that stagnation allows the water temperature to rise in the service line which increases the activity of the bacteria suspended and attached in the service line, which are in most cases consuming substrate (BDOC) and multiplying. Increased temperature also increases the rate of reaction of chlorine, which is depleted as it inactivates the bacteria and oxidizes organic and inorganic constituents in the water column. These reactions all occur concurrently.

Prevost et al. (1997) also indicated that there appeared to be a "plateau value" at which bacterial growth in the dead ends would approach a limit, regardless of residence time. This could be due to the limited amount of substrate and nutrients available in the line.

Carter et al. (1998) studied a dead end loop in a distribution system. A dead end loop can be described as a section of distribution main that is fed from two ends by the same feeder pipe, similar to what might be constructed on a Crescent. While residual concentrations entering the loop varied widely throughout the study period, the chlorine residual was found to reach minimum values as water stagnated at or near the apex of the loop. Carter et al. (1998) indicated that due to the variability of the residuals, periodic grab samples would likely not have identified the periods of poor quality.

Further concern over dead end pipes is generated by the practice of using small diameter pipes in these locations. Chlorine residual has been observed to decay quicker in smaller diameter pipe than larger pipe given the same initial chlorine concentration (Prevost et al., 1998), largely because there is more surface area per unit of water (and thus more adhered material to react with chlorine) as compared to larger pipes. This larger surface area to volume ratio favours faster chlorine decay by increasing the number of chlorine consuming elements: biofilm, organic deposits at the pipe surface, and pipe material (Prevost et al., 1997).

2.3.8 Biofilms

The term 'biofilm' is used to describe a layer of microorganisms in an aquatic environment held together in a polymeric matrix attached to a substratum such as pipes, tubercles, or sediment deposits (Momba et al., 2000). Momba et al. (2000) also stated that biofilms on surfaces exposed to drinking water in distribution systems may well be the main source of planktonic (suspended) bacteria since up to 1000 viable organisms may be present for each planktonic cell detected. This claim may have been based upon the work of Van der Wende et al. (1989), Piriou et al. (1998) and Laurent et al. (1993), who all noted an order of magnitude difference in the number of suspended and attached bacteria with the latter being the larger. Based on their own research, other investigators (Ridgway and Olson, 1981; Prevost et al., 1998) have suggested that due to attachment, enumeration of suspended bacteria is a gross underestimation of the actual number of bacteria present in water supply systems.

Established biofilms are often difficult to remove from the walls of the pipe due to the strength of the bond formed with the substratum, given adequate time. Based on the above noted findings, the focus of water quality preservation should shift from control of suspended bacteria to the control of biofilms in water distribution.

These attached microcolonies present a danger to users of drinking water systems because:

1. High numbers of HPC bacteria can be released into the bulk water as a portion of the film is sloughed off due to shear forces created by flow or death of the organisms at the pipe surface (Brading et al., 1995);
2. Biofilms have been shown to protect bacteria from the effects of oxidants (LeChevallier et al., 1988a);
3. Biofilms are known to harbour and provide an environment conducive to the multiplication of pathogens and opportunistic pathogens such as *Pseudomonas, Mycobacter, Campylobacter, Klebsiella, Aeromonas, Legionella, Salmonella typhumerium* (Momba et al., 2000), *Helicobacter pylori* (Mackay et al., 1998), *Escheria Coli* (Fass et al., 1996);

4. Biofilms and the compounds associated with them can create a significant chlorine demand, depleting residuals which might allow re-growth elsewhere in the system (Van Der Wende et al., 1989; Lund and Ormerond, 1995);
5. Biofilms can encourage corrosion and affect the hydraulics of a distribution system (Percival, 1998); and
6. Sloughing or metabolism of attached bacteria can create taste, odour, and colour problems (Percival, 1998).

Bacteria have an affinity for surfaces. Some microbiologists have hypothesized that most bacteria in aquatic environments exist at solid-liquid interfaces (LeChevallier, 1990). Marshall (1979), as quoted by Percival (1998), described the surface in flowing water systems as a 'relatively nutrient-rich haven in an otherwise low nutrient environment'. Percival (1998) provides a detailed review of potable water biofilms in which the stages of biofilm development are discussed. The summary of these stages as presented by Percival (1998) is as follows:

1. Development of a surface conditioning film, i.e. transport of organic molecules to the wetted surface;
2. Transport by which the organism is brought into close proximity of the pipe surface;
3. Adhesion;
4. Growth and division of the organisms with the colonization of the pipe surface, microcolony formation, and biofilm formation; and
5. Detachment.

Figure 2.4 shows the schematic representation of the interaction of suspended and attached bacteria, substrate, and oxidant. The following sections discuss the mechanisms of biofilm formation, utilization of substrate, interaction with oxidants, effect of pipe material, as well as the mechanisms causing detachment, and the potential for deterioration of water quality due to the presence and persistence of biofilms in drinking water distribution systems.

2.3.8.1 Formation

2.3.8.1.1 Transport and Conditioning Film

The formation of biofilms in potable water supplies is a complex, yet naturally occurring phenomenon. Several authors have hypothesized that the conditioning film that forms on the interior surface is a product of the tendency of submerged solid surfaces to assume a net negative charge, which then attracts cationic dissolved minerals and organics to it (Brading et al., 1995). Ridgway and Olson (1981), through scanning electron microscope technology, have reported this conditioning film as containing a complex network of deep fissures and cavities. This greatly increases the total surface area of the pipe, providing potential microhabitats for organisms to invade and colonize. Percival (1998) notes that the transport of cells to the surface of the pipe could be a rate-controlling step in the formation of biofilms.

2.3.8.1.2 Attachment

Both suspended and attached bacteria produce excretions composed mainly of polysaccharides commonly referred to as glycocalyx, or as it will be referred to herein, extracellular polymeric substances (EPS). The nature of EPS imparts the slimy texture of biofilms (Momba et al., 2000).

Adhesion is aided by the production of EPS, which can be a type of 'cementing' substance employed by bacteria (Percival, 1998). Other investigators (Ridgway and Olson, 1981; Brading et al., 1995) noted some bacteria use filaments and extracellular fibrils to anchor themselves into cracks in the conditioning film.

Once attached, the bacteria produce large amounts of EPS, presumably for the entrapment of nutrients and protection from the surrounding environment. Biofilm can be approximately 50 to 90% EPS (Percival, 1998). Formation of the EPS layer also contributes to the capture of other species of bacteria, which may be why mature biofilms have been shown to contain a diverse population of bacteria (Brading et al., 1995).

The benefits provided to biofilm organisms by the production of EPS is described by Percival (1998):

1. Cohesive forces within the biofilm;
2. Adsorption of nutrients;
3. Protection of immobilized cells from the effects of rapid environmental changes;
4. Adsorption of heavy metals from the environment;
5. Adsorption of particulate matter;
6. A means of intercellular communication; and
7. Enhancement of intercellular transfer of genetic material.

Not noted by Percival (1998) is the resistance EPS and molecules trapped in EPS may provide to chemical disinfection (Butterfield et al., 1999). Nutrient adsorption and oxidant resistance will be discussed in subsequent sections.

2.3.8.2 Substrate Uptake

LeChevallier (1990) describes the benefit of attachment for organisms in water distribution systems. He notes that the low nutrient concentration in water when combined with high flow rates can transport large masses of nutrients to fixed organisms. Additionally, LeChevallier (1990) notes that the production of EPS may also be a factor in nutrient capture. Volk and LeChevallier (2000) also note that fixed bacteria are more active than their suspended counterparts because co-metabolism can occur between species.

Christensen and Characklis (1990) suggest that the anionic character of EPS give biofilms cation exchange properties. They suggest that bacteria can trap and concentrate cationic nutrients in this way. The volume and composition of EPS has been reported to change with the concentration of available substrate and growth phase. Wrangstadt et al. (1986) noted that the starvation of cells caused extensive release of a viscous polysaccharide not observed in the normally growing cells. This may be an attempt to trap more nutrients or protect the organism from damage.

2.3.8.3 Studies of Biofilm Substrate Utilization

Laboratory research by Van der Kooij et al. (1995) showed a threshold substrate concentration at 0.1 μg-C/L for the formation of biofilms. Van der Kooij et al. (1995) did not expect that the situation observed in controlled laboratory conditions was likely in a distribution system but noted that it indicates that bacterial populations can thrive at extremely low levels of substrate.

2.3.8.3.1 Bench/Pilot Scale

Several investigators have utilized laboratory devices to study the effect of substrate concentration on biofilm formation (Van der Wende et al., 1989; Ollos et al., 1998; Butterfield et al., 1999; Ollos et al., 2003). The experimental setup was composed of annular reactors consisting of a rotating drum placed inside a pipe sleeve. Flow rate through the reactors is controlled to simulate different HRTs in a distribution system. The speed of the rotating drum can be adjusted to simulate the shear conditions present in a distribution system. These investigators feel that these continuously stirred tank reactors are representative of a finite portion of a distribution system. Similar results were obtained by each of the investigations. In general, biofilms proliferated in conditions where substrate was available and decreased in density as the substrate was consumed.

The effects of shear were noted by Ollos et al. (2003). In the presence of low substrate concentrations in natural water, the biofilm densities were higher with increased shear. This might have been a result of increased turbulence resulting in increased substrate transport to the biofilm surface.

Van der Wende et al. (1989) used four annular reactors in series to simulate several different sections of a distribution system. They too found an increase in bacterial populations with increased nutrient loading. For different loading rates, the researchers found that the maximum populations were attained at approximately the same residence time and either remained constant or decreased with increased residence time. This suggests that a nutrient limitation can limit the maximum biofilm population.

Butterfield et al. (1999) ran two reactors fed with the same source water. One of these source waters was given a chlorine residual to allow direct comparison of the effect of chlorine on substrate uptake. They observed that the specific growth rate and substrate uptake for the unchlorinated water were lower than in the chlorinated reactor. Although the growth rate and substrate uptake were higher, Butterfield et al. (1999) observed that the yield of the chlorinated reactor was lower, meaning less biomass was produced. They attributed this anomaly to the chlorine reducing the EPS and the bacteria trying to replace it. Specifically, they noted the chlorinated biofilm, growing at rates faster than the unchlorinated biofilm but at a lower observed yield, may have required more substrate for metabolism, repair and the manufacture of polysaccharides and proteins for EPS (Butterfield et al., 1999). Note that the biofilm growth was not stopped, only controlled by the presence of free chlorine. The main criticism of laboratory experiments is that they are often not representative of real networks and they focus on specific influences rather than interactions between parameters (Hallam et al., 2001).

Other experiments with active biomass have been conducted through the use of pipe loops (Piriou et al., 1998; Block et al., 1994; and Mathieu et al., 1993). The arrangement of the experimental pipe loops are similar. In the case of Piriou et al. (1998), the control water enters the network and is circulated through the loop to achieve the desired velocity and residence time. The loops following have the same arrangement only the feed water comes from the loop prior. Temperature can be controlled by means of a heat exchanger or cooling system. For the research of Block et al. (1994) and Mathieu et al. (1993) the pipe loop was again a three-loop series but without re-circulation or temperature control.

Piriou et al. (1998) reported an increase in biofilm activity with increased substrate concentration. Above a concentration of approximately 0.4 to 0.5 mg/L BDOC, the investigators also noted that the biofilm activity reached a plateau, indicating a nutrient uptake limitation.

The influence of chlorine in the presence of a BDOC concentration of 0.4 mg/L was also investigated. The measurements taken were for suspended bacteria so biofilm

activity cannot be analyzed directly but can be inferred from the reaction of the suspended bacteria. Above a free residual concentration of 0.2 mg/L , suspended bacteria slowly declined, indicating little change in fixed activity. When the residual went below 0.2 mg/L , there was an increase in activity. After 12 hours the free chlorine was less than 0.05 mg/L , and the HPC levels were almost as high as the unchlorinated water. This showed the ability of biofilms to grow quickly in the presence of low levels of chlorine.

Block et al. (1994), using a series of three pipe loops, fixed the feed water conditions and let nature take its course. They reported trends similar to those observed by Piriou et al. (1998).

2.3.8.3.2 Full Scale Networks

Niquette et al. (2001) studied the depletion of BDOC in parts of a Brussels distribution system. The results following consumption were plotted versus initial BDOC. There appeared to be no consumption in the network below an initial BDOC concentration of 0.25 mg/L , indicating growth was limited. Niquette et al. (2001) also noted that this threshold concentration was in the presence of a free chlorine residual of 0.07 mg/L. Laurent et al. (1993) reported similar results. It was noted that in the presence of a free chlorine residual, fixed biomass was limited but no values for the residual were given.

In view of these studies it seems then that some combination of chlorine residual concentration and organic matter content (substrate) may be the key to limiting the proliferation of biofilms.

2.3.8.4 Effects of Oxidant

LeChevallier (1990) proposes that attachment of bacteria in biofilms is beneficial for bacteria because they are only exposed on one side to the effect of an oxidant and that exposure is limited by physical and transport phenomena related to the EPS.

Maintenance of a residual oxidant in the bulk water of a distribution system has been proven effective for inactivation of suspended bacteria (originating from biofilms)

provided the ratio of substrate and oxidant residual concentrations are adequate. The maintenance of an oxidant residual at levels that do not illicit taste and odour complaints is typically not high enough to reduce the biofilm activity in the absence of nutrient control. Often, if any penetration into the biofilm is to be achieved, an abnormally high level of oxidant must be used in order to penetrate the EPS surrounding the bacteria. Organic and inorganic particulate bound by the biofilm can exacerbate the demand EPS puts on oxidant residuals. This section will discuss the research related to the penetration of oxidants into biofilms.

2.3.8.4.1 Biofilm Penetration by Oxidants

The effects of an oxidant on the distribution biofilm is dependent on several factors. These factors typically include the oxidant type (free chlorine or monochloramine) nutrient conditions, age of the biofilm (level of attachment) and substratum material.

Some investigators have determined that low nutrient conditions can increase the resistance of biofilm bacteria to the effects of chlorine. LeChevallier (1988a) noted a 3 to 4-fold increase in resistance to chlorine over the same species grown in a high nutrient environment. LeChevallier (1988a) noted that age of the biofilm and encapsulation also increased resistance of the organisms to free chlorine. LeChevallier (1988a) found the combined effects of these resistances were multiplicative, and an attached species grown under low nutrient conditions could be up to 600 times more resistant to free chlorine than its suspended counterpart.

De Beer et al., (1994) used annular reactors (apparatus described in Section 2.3.8.3.1) to culture biofilms to a thickness of 150 to 200 μm on stainless steel slides. Using chlorine sensitive microelectrodes, the researchers measured the chlorine concentration as it penetrated the biofilms. The researchers hypothesized that if the chlorine was only diffusing into the biofilm, the concentration of chlorine in the biomass at the biofilm-solid interface would eventually reach the same level observed in the bulk water. This was not the case. Chlorine, in every profile shown, was reduced as it penetrated the biofilm, indicating there was a reaction taking place. Another discovery

of note, which may also affect disinfection of biofilms in full-scale networks, was that there appeared to be a boundary condition present near the biofilm-bulk water interface. Chlorine concentration was observed to decrease before it came in contact with the biofilm. This could be due to the boundary layer that exists at the liquid-solid interface. This layer would likely be smaller in quickly flowing waters but it may still have an effect on the diffusion of chlorine from the bulk water to the surface of the biofilm.

LeChevallier et al. (1990a) compared the effect of substratum material and found that for smooth pipes, such as copper and PVC, free chlorine is more effective in inactivating attached bacteria than monochloramine.

In the case of iron pipes, chloramines have been proven more effective than free chlorine in penetrating biofilms. This difference in efficacy was attributed to the stability of chloramines, which do not react with as many compounds as the hypochlorous acid present in free chlorine.

Neden et al. (1992) compared three areas of the Greater Vancouver Water District (GVWD). The first area did not carry a supplemented oxidant residual, the second was downstream of a secondary chlorination facility, and the third was downstream of a station that injected chloramine. Within these three areas, the GVWD installed biofilm monitoring stations consisting of segmented observation pipes constructed from PVC, ductile iron, and cast iron. During the two-year study, Neden et al. (1992) noted that chloramines provided superior control of attached organisms over the other oxidants for each of the pipe types tested.

The relatively poor performance of free chlorine can be attributed to the fact that it is highly reactive with many materials. The EPS layer of a biofilm is composed of organic compounds that would react with chlorine. Inorganic and organic molecules trapped within the biofilm matrix would also react with the free chlorine. Therefore, in cases where water distribution lines carry and distribute chlorine reactive substances, chloramine is likely better for controlling biofilms. In general, the works cited indicate a wide variety of oxidant concentrations were found to control biofilms in different situations.

Lu et al. (1999) reported that when the pH increases, the chlorine decay kinetic coefficient decreases. On newly established biofilms, Lu et al. (1999) noted an increased chlorine decay rate for lower pH, presumably due to the higher concentration of HOCl at lower pH. For more established biofilms the decay rate of chlorine is less dependent on pH, suggesting it becomes equally difficult for hypochlorous acid and hypochlorite ions to penetrate the biomass. This difficulty likely stems from the well-established EPS protecting the biofilm.

The conclusion that can be drawn is that there is no universal solution for inactivation of biofilm bacteria. The complications presented by biofilm age, substrate and inorganic ion concentrations, as well as pipe material, must be considered when choosing the type and concentration of oxidant for controlling biofilms.

2.3.8.4.2 Biofilm Control with Residual Oxidants

Although biofilms are difficult to obliterate, they can be controlled in some cases. Oxidant residuals do have an effect on the attached bacteria but this control is often indicated by inactivation of suspended bacteria released from the biofilm.

A number of researchers have investigated the levels for control or reduction of biofilm populations, including Hallam et al. (2001), Ollos et al. (1998), Niquette et al. (2001), Mathieu et al. (1993), Block et al. (1994), and Gatel et al. (1998). These researchers reported a wide variety of residuals and results for each of their situations.

Van der Wende et al. (1989) noted that in their annular reactors the number of suspended bacteria at low chlorine concentrations was approximately the same as what was recorded for unchlorinated water. This suggests a threshold level. A higher initial chlorine concentration only served to reduce the bacterial densities at low HRT. The low values of chlorine resulting at higher HRT were followed by a dramatic increase in bacterial numbers.

In the presence of adequate substrate, it appears that the maintenance of an oxidant residual will often only delay the formation of a biofilm and may only affect the

spatial distribution of biofilms. Lund and Ormerond (1995) and Gatel et al. (1998) also observed this delayed accumulation.

2.3.8.5 Temperature Effects

Ollos et al. (2003) presents information on the formation of biofilms as a result of temperature. Experiments were conducted at 8 and 26°C for supplemented (500 μg/L sodium acetate for carbon substrate) and unsupplemented waters. They reported that the biofilm density in unsupplemented water at 26°C was nearly twice that at 8°C. In supplemented water the biofilm density at 26°C was approximately 10 times greater than that observed at 8°C.

Hallam et al. (2001) noted that biofilm formation potential is temperature dependent. A decreased formation potential with lower temperature, measured by enumeration of biomass formed on spherical media within a reactor, was recorded for samples taken at 15°C and 5°C with similar chlorine residuals.

Lund and Ormerond (1995) observed an increase in biofilm production during a period when their experimental apparatus was off-line and temperatures rose from 5°C to approximately 10°C. This increase in biofilm production was also noted as temperatures varied throughout the year. The highest increases in biofilm density coincided with peak temperatures in August and September.

The study by Lund and Ormerond (1995) found that, in the presence of a free chlorine residual of 0.05 mg/L , no biofilm was detected in the water at a temperature of about 5°C. This is not to say that re-growth cannot occur at low temperatures. In colder climates, where the average water temperature was approximately 10.6°C, an increase in bacterial activity was evident starting at 5°C (LeChevallier et al., 1996b). This trend was attributed to the possibility that microbes in the water had adapted their growth cycle to the low temperatures. Emde et al. (1992) reported the colonization of pipe tubercles in the distribution system of Yellowknife, NWT. Despite the fact that the water temperature was maintained near 0°C, Emde et al. (1992) recovered and identified 15 different bacteria and fungi types from the water column and 27 different bacteria

and fungi types from the pipe tubercles, some of which were identified as opportunistic pathogens.

2.3.8.6 Affinity for Pipe Materials

Many authors have investigated the effect of substratum material on the formation of biofilms. These studies reported variable results. Table 2.6 summarizes the literature reviewed with respect to potential for biofilm formation. In the table the material having the most potential for supporting biofilm will have the highest number. Conversely, the material with the lowest potential will have the lowest number.

Although the formation potential of synthetic materials are believed by some to be the highest, the attached biomass in the presence of a oxidant residual has been shown to decrease below that of non-synthetic materials. Hallam et al. (2001) recorded that at a low chlorine residual, PVC had a higher biofilm potential than glass. However, the presence of 0.3 mg/L free chlorine residual resulted in the glass having a higher formation potential than PVC. Neden et al. (1992) found that PVC supported lower densities of attached bacteria than unlined cast iron or lined ductile iron. This data was collected in an operating distribution system maintaining an oxidant residual. Niquette et al. (2000) reported similar findings on coupons placed in a distribution system with a chlorine residual.

The corrosion of pipe surfaces may also contribute to the formation of biofilms. Pipe tubercles can create quiescent zones or crevasses in which bacterial populations would be free of the effects of oxidants. LeChevallier et al. (1987) stated that the irregular surface and production of EPS can protect the bacteria from the effects of an oxidant.

Van der Kooij et al. (1999) have questioned the biostability of synthetic pipes. Additives used in the materials and manufacturing process may provide a source of substrate for attached bacteria. In the Netherlands, where substrate limitation is used in lieu of oxidant residuals to control re-growth, synthetic pipes may be an issue. In North America, where treated waters often carry some substrate and an oxidant residual is required by law, the contribution of pipe material to the total substrate concentration is likely minimal.

Table 2.6 Potential for biofilm formation as a function of substratum material.

Source	PE	PVC	CI	DI	CL
Niquette et al. (2001)	1	2	4*	n.c.	3
Hallam et al. (2001)	2	3	n.c.	n.c.	1
Van der Kooij et al. (1999)	2	1	n.c.	n.c.	n.c.
Schwartz et al. (1998)	3	2	n.c.	1	n.c.
Neden et al. (1992)	n.c.	1	3	n.c.	2

Legend: PE – polyethylene, PVC – polyvinyl chloride, CI – cast iron, DI – ductile iron, CL – cement lined, n.c. – not considered.
* Note: Material with highest potential for biofilm formation is assigned the highest number.

2.3.8.7 Water Quality Deterioration Potential due to Biofilms

2.3.8.7.1 Sloughing Mechanisms

The largest potential for water quality deterioration due to biofilms is the release of bacteria into the bulk water column by sloughing. There are several mechanisms that can cause biofilm sloughing including shear, abrasion, death and chlorination. A biofilm in equilibrium has a growth rate equal to the loss of bacteria due to these mechanisms. Nutrient loading is relatively constant due to the water flowing through a distribution system. Thus a finite mass of biofilm at equilibrium could very well seed the remainder of the network through sloughing.

Brading et al. (1995) describe detachment under the influence of shear as a result of thinning of the boundary layer by turbulence at high flow rates. As the boundary layer thins, more of the biofilm may project into the turbulent bulk water and is, in essence, eroded. Abrasion by suspended particles in the water column striking the biofilm can also cause detachment. Brading et al. (1995) note that older biofilms may be more dense and likely have increased resistance to the effects of abrasion. Percival (1998) describes mature biofilms as having a nutrient gradient. This gradient develops

with aerobic respiration at the upper surface, fermentative bacteria and then anaerobic bacteria at the surface of the pipe (i.e. the bottom of the biofilm). If nutrients are limited or oxygen transfer is insufficient, parts of the biofilm can die resulting in a loss of cohesion in a portion of the biofilm. This weakened portion may be prone to detachment, carrying the live cells into the bulk water with the dead. Brading et al. (1995) have also noted that detachment could be caused by rapid growth of the biofilm. They implied that rapidly growing bacteria do not attach well. Another cause for detachment may be competition between species. Gantzer et al. (1989), cited by Brading et al. (1995), suggested that some bacteria may de-polymerize the EPS of other species or be incompatible with others EPS, weakening the biofilm and creating a flaw at which sloughing could start.

Chlorination of biofilms can cause massive sloughing due to destruction of the EPS matrix. White (1986), as cited by LeChevallier et al. (1988b), describes complications related to the use of free chlorine to control biofilms:

> *"Although free chlorine has been successful for control of bacterial biofilms in many cases, new problems, including temporary production of chlorinous tastes and odours and complaints due to dirty water caused by sloughing of debris, were encountered in some cases when a free chlorine residual was used."*

2.3.8.7.2 Examples of Distribution System Biofilms

Percival (1998) describes a case of biofilm contamination, which occurred in Queensland, Australia. A biofilm had formed within the distribution network and the primary species present was *hyphomicrobium*. The cell wall of *hyphomicrobium* acts as a catalyst for oxidation of the manganese present in the water at low concentrations. The hypha-like structures these bacteria form were coated in oxidized manganese. When the biofilm would slough, the black precipitate of manganese oxide would be released, creating black water.

LeChevallier et al. (1987) describes chronic bacteriological problems experienced by a drinking water utility. In 1984, the utility isolated coliform bacteria from the distribution system. Varied levels of coliforms where recorded thereafter. Although samples of the water treatment plant effluent had not indicated significant levels of coliform organisms, the utility believed this was a failure of the treatment

process, and took extensive measures to attempt to rectify the problem. The utility increased its sampling program and analyzed nearly 800 samples for injured coliforms, all of which were uniformly negative.

LeChevallier (1987) indicated this suggested the bacterial breakthrough was not occurring at the treatment plant. In addition to the measures taken at the water treatment plant, the distribution residuals were increased, alternate oxidants were tried, and the distribution system was systematically flushed, to no avail.

LeChevallier (1987) noted that almost 40 % of the 500 distribution system samples collected during May through August of 1986, were positive for coliforms and had varied counts. The largest increase in coliform counts was observed between the water treatment plant and the study site 1.1 km away. Some 235 coliform bacteria types and over 80 HPC types were identified indicating a wide diversity of species typical of established biofilms. Many of the species identified in the water column were later found to be present in the biofilm. Hydraulic modelling of the system determined that the HRT from the water treatment plant to the study site was at most 102 minutes. To achieve the bacteria counts observed at the study site by reproduction in the bulk water column the coliform bacteria would have to replicate every 30 minutes, which was determined to be impossible under low temperature, low nutrient and high oxidant residual conditions. LeChevallier (1987) concluded that the bacteria must have been originating from biofilm within the trunk pipeline. LeChevallier (1987) calculated that the AOC concentration utilized in the main between the water treatment plant and the study site was sufficient to support 80,000 organisms/mL, which was similar to the levels observed within the study site. Statistical modelling showed that most of the water quality deterioration could be related to the dead end sampling locations. LeChevallier (1987) went on to state that these results emphasize that water stagnation, even in short segments of the distribution system, can have profound and adverse effects on potable water quality.

2.3.9 Summary of Factors Controlling Biofilm Growth

Biofilm formation can occur at very low levels of substrate. Oxidant residuals are not able to completely remove a biofilm, but have been shown to reduce its production. Dead ends have been shown to severely affect the quality of treated water. Operational and maintenance issues have been shown to be a potential cause of contamination or can temporarily correct HRT issues, particularly in dead ends. Increases in temperature have been noted to exacerbate water quality deterioration. In the context of an existing rural water pipeline, configuration is beyond control of the utility, as is temperature, and flushing and swabbing of lines have been shown to be temporary solutions to larger issues. With the factors beyond the control of a utility it then appears that some combination of oxidant residual and reduction of organic matter concentration is the key to limiting re-growth.

Gatel et al. (1995) noted a relationship between chlorine concentration and substrate uptake in a study of 41 distribution systems in France. More specifically, the study showed that when the chlorine residual was approximately 0.29 mg/L , BDOC was only consumed in the distribution network if its concentration was above approximately 0.2 mg/L . When the chlorine residual was approximately 0.42 mg/L , BDOC was only consumed in the network when its concentration exceeded approximately 0.6 mg/L . The variation in the threshold concentration for chlorine noted by other investigators could have been due to the effect of BDOC reported by Gatel et al. (1995). Piriou et al. (1998) reported higher rates of re-growth for lower chlorine residual in the presence of the same concentration of substrate. Volk and LeChevallier (2000) reported that the threshold values for coliform occurrence were a temperature > 15°C, a free chlorine residual < 0.5 mg/L and an AOC concentration > 100 µg/L . Some 75 % of the positive coliform samples identified in their study were recorded when two or three of these criteria were exceeded in a distribution system. This supports the philosophy that no one factor is responsible for re-growth, rather it is a combination of effects.

Ollos et al. (2003) demonstrated the effects of chlorine on biomass in annular reactors for different levels of substrate. For unsupplemented water, the fixed biomass

counts in the presence of a 0.5 mg/L chlorine residual were approximately 2.5% of original counts, developed in unchlorinated water. Addition of substrate increased the bacterial density to approximately 5.8 $\times 10^6$ CFU/cm^2 in unchlorinated water. The presence of a 0.5 mg/L chlorine residual reduced the viable counts by 99.9%. Despite the large reduction, the supplemented water contained more than four times the bacterial population of the unsupplemented water. It should be noted that the supplemented and unsupplemented tests were performed at 26 and 8°C. Temperature was shown to have an effect on supplemented samples, however the relationship remains the same in that higher substrate levels for the same chlorine concentration result in higher biofilm densities.

It should be noted that limiting re-growth is not preventing bacterial growth in a system. Bacteria will still grow, die and/or be released from biofilms in the presence of chlorine and with the limitation of available substrate. A system in which re-growth is limited is where an equilibrium between biofilm bacterial numbers, substrate concentration and chlorine residual is established. This equilibrium is such that:

1. The amount of substrate utilized in the system between two points is below the limit of detection;
2. The chlorine is present in sufficient concentration that bacteria released from the biofilm are inactivated to below regulatory limits;
3. The substrate from the water column and cell lysis is limited such that the growth of biofilm organisms is equal to the death of the organisms through starvation, age, and inactivation by chlorine.

Values for the parameters controlling biofilm growth presented herein have been demonstrated to vary between systems. While the threshold values for these systems may be similar, no two systems have the same contributing factors to the growth of biofilms. Thus, in the investigation and control of biofilms, care must be taken to examine the contributing factors before applying a solution.

Based upon the research of many of the aforementioned investigators, the suggested threshold guidelines for controlling biofilm growth are as follows:

1. Temperature <15°C;

2. Free chlorine residual > 0.1 mg/L as Cl_2;
3. BDOC < 0.2 mg/L ; and
4. AOC < 10 µg/L .

2.4 Modelling

The use of computer models has revolutionized the analysis of piping systems. Often the approach taken to determine head loss and flow rates in looped systems has involved the use of iterative calculations. Repetitious calculations are easily performed by computers at a fraction of the time it takes to perform them manually. Thus, the use of computers in network modelling has increased speed of analysis. Parameters and the effects of their change can be analyzed quickly and efficiently between scenarios, and in some cases, the software interface allows a side by side comparison of alternate scenarios. The following sections provide a brief overview of some of the modelling programs available for the analysis of hydraulic, chemical and biological constituents in water distribution systems. Discussion of hydraulic modelling software is based largely on the author's experience with the two programs identified.

2.4.1 Hydraulic Modelling Software

Hydraulic models consist of several basic components. Links are often used to represent pipes; nodes represent fittings or changes in elevation or direction. Many different types of pumps, tanks and valves can also be represented in the model. Logic based control using time, pressure, flow and status of other components can often be incorporated to dictate position and status (on/off) of actuated valves and pumps (Rossman, 2000). Head loss characteristics can be attached to valves and pump curves can be specified to dictate the operating range of the pumps. The size and operating levels of storage reservoirs or tanks can also be specified. Transport and decay of parameters such as chlorine residual can be incorporated into the models (Rossman, 2000). The age of a volume of water can be tracked to determine the HRT at any point in the system. Most importantly, the user demand can be specified for each node, including time based variations. These variations are created by assigning a demand

pattern table to nodes to reflect the usage patterns of the consumers (Haestad Methods et al., 2003). The general setup and development of network hydraulic models is described by Rossman (2000) and Haestad Methods et al. (2003).

One of the most common commercial software package used by water professionals is WaterCAD from Haestad Methods. This package allows creation of a model network within AutoCAD. The input parameters can be specified and include fluid type, temperature, material, solution algorithm, diameter, and demands (Haestad Methods et al., 2003). The interface allows the user to easily specify changes within the physical and operational parameter database through the use of filter and mathematical functions and generate presentation quality charts of hydraulic gradeline, head loss, pressure, flow rate and velocity (Haestad Methods et al., 2003). Recent additions to the software include an automated calibrator, which has been credited with reducing the iterative process of network calibration (Haestad Methods et al., 2003). The user-friendly interface of WaterCAD has allowed inexperienced modellers to create and run network simulations quickly and easily. This is also where the danger lies. Use of the program and calibration software can produce results but inexperienced users are often unable to determine if the results are credible. In short, many assume that because it came from a computer the results must be correct, which is not always the case.

The major drawback of WaterCAD is the price. Licenses are issued on the basis of the number of pipes (links) the package is able to create and analyze. Because of its popularity and widespread use, unlimited pipe licenses for WaterCAD can be up to $10,000 (U.S.) before addition of the calibration or GIS packages. The use of WaterCAD is then often limited to consultants who can use the fees charged to clients to offset their investment in the software.

The USEPA has issued a free software package for analyzing pipe networks called EPANET. EPANET Version 2.0 is a notable improvement over the initial version. The first version of EPANET required the input data to be entered within a text file, which was time consuming and often confusing. EPANET 2.0 has been upgraded to be a windows interfaced modelling program, allowing creation of the network model on screen with drawing tools (Rossman, 2000), similar to the WaterCAD stand-alone

program (no AutoCAD interface). The drawbacks to EPANET are the less user-friendly aspects of the interface over WaterCAD. The user can still input the data for each item in the program but the data table for changing the parameters contains fewer filters and mathematical functions than the WaterCAD program. This means that more effort is required by the modeller, particularly when a change in demand or demand patterns is required. The solution capabilities of EPANET are similar to those in WaterCAD, and some consultants have used EPANET to allow their clients to view and use the model they have created without the additional cost associated with a software purchase. Although model creation, calibration and changes are more difficult with EPANET, the fact that it is available at no charge far outweighs the benefits of WaterCAD for a utility using modelling software for analysis purposes, assuming an experienced modeller is on staff. If the user were to use WaterCAD to generate profits through the modelling work done, the cost of purchasing might be offset. Cost was the determining factor in the selection of EPANET for use in modelling the rural water pipeline monitored in this study.

2.4.2 Biological Water Quality Modelling Software

Some of the factors affecting free chlorine decay have been identified as dissolved organic carbon concentration, reaction with attached biomass, and inactivation of suspended bacteria. The reactions are not considered in programs such as EPANET or WaterCAD. In addition, calculation of biological activity such as the production of suspended and attached biomass is beyond the capabilities of these programs. Dombay et al. (1999) note that due to water quality changes within a distribution network, the design and operation of drinking water distribution systems cannot be considered only from the hydraulic view, but reactor theory and process engineering measures should be integrated. Dombay et al. (1999) note further that to understand and describe bacterial re-growth phenomena in drinking water systems, deterministic modelling is one of the most appropriate tools available to researchers to test hypotheses, and to engineers to implement water quality management in the network. Two computer modelling programs, which consider these effects, have been developed by researchers. These are the SANCHO (Servais et al., 1995) and PICCOBIO (Piriou et al., 1998) models.

The SANCHO model was developed based on relationships (determined through research by the authors and others) between organic matter, free chlorine and fixed and suspended bacteria. The model incorporates the mathematical relationships, that describe the kinetics between these parameters, specifically:

1. Interaction between organic matter and bacteria including the utilization of substrates, the growth of free and fixed bacteria (including temperature effects), and bacterial mortality that releases organic matter;
2. Reversible adsorption and attachment of bacteria to the pipe surface; and
3. Chemical consumption of the free chlorine by organic matter and the impact of free chlorine on bacterial activity and mortality.

The model output was subsequently compared to data collected from distribution systems in France and Quebec and showed adequate agreement. It should be noted that the model is only constructed for steady state conditions.

Laurent et al. (1997) conducted simulations using SANCHO to determine the effects of different water quality parameters at the input of the system, the findings of which were similar to that reported by Servais et al. (1995). Laurent et al. (1997) noted that the presence of a free chlorine residual, when coupled with adequate substrate, only delayed biofilm formation until such a time that the chlorine residual was low enough to permit biological activity. The magnitude of re-growth was noted to decrease in the presence of a higher initial free chlorine residual. The effect of BDOC on re-growth was simulated and showed a threshold value of approximately 0.15 mg/L below which no re-growth was expected to occur, even in the absence of a free chlorine residual. Authors investigating full-scale distribution networks (discussed previously) have reported similar threshold values.

The PICCOBIO modelling program is similar in nature to the SANCHO model. It considers the same interactions between parameters but differs in the methods of calculation and steps involved in the reduction of BDOC to substrate and definitions of free and fixed bacteria. The largest difference between the two programs is that PICCOBIO is coupled with the PICCOLO hydraulic computer model, which may better describe the propagation and spatial variation of re-growth in a distribution system.

Piriou et al. (1997) and Piriou et al. (1998) described the verification of this model on data collected from the city of Marseille in southern France, and under the controlled conditions of an experimental pipe loop, respectively. Piriou et al. (1998) also simulated a pipe system to assess the influence of BDOC, temperature and free chlorine residual. The findings, which have been noted previously in this review, were that biofilms could be controlled if the temperature remains below 15°C, the BDOC concentration is less than 0.2 mg/L , and the free chlorine residual is above 0.1 mg/L as Cl_2.

These models represent a powerful tool for investigating water quality deterioration due to biofilm proliferation. Unfortunately, these programs are proprietary and were not made available for incorporation into this study. As a result only EPANET was used in conjunction with spreadsheet calculations for estimating chlorine decay coefficients in the modeled network.

2.4.3 Capability of EPANET

The following sections provide a brief overview of the capabilities of EPANET 2.0, used in modelling the rural water pipeline chosen in this study. A full description of the software capability and instructions for its use can be found in Rossman (2000).

2.4.3.1 Hydraulic Capability

Rossman (2000) outlines the capabilities of the EPANET hydraulic analysis as follows:

1. No limit on the size of the network that can be analyzed;
2. Computes friction head loss using the Hazen-Williams, Darcy-Weisbach, or Chezy-Manning formulas;
3. Includes minor head losses for bends, fittings, etc.;
4. Models constant or variable speed pumps;
5. Computes pumping energy and cost;
6. Models various types of valves including shutoff, check, pressure regulating and flow control valves;
7. Allows storage tanks to have any shape (i.e. diameter can vary with height);

8. Considers multiple demand categories at nodes, each with its own pattern of time variation;
9. Models pressure-dependent flow issuing from emitters (sprinkler heads); and
10. Can base system operation on both simple tank level or timer controls and on complex rule based controls.

EPANET uses iterative calculations to determine flow and pressure variations in network with variable demand. The method used to solve and update the iterations is called the gradient method. A detailed description of the method can be found in the appendix of Rossman (2000).

Rossman (2000) describes the gradient method in general terms. The solution begins with an initial estimate for flows in each pipe that may not necessarily satisfy flow continuity. For each of the iterations in the method the head at each node is found by solving a matrix equation. After the new head is found the new flow for each pipe is computed using a correction involving the calculated heads at the upstream and downstream end of the link, a flow correction factor, and the inverse derivative of the head loss between the upstream and downstream ends of the link all subtracted from the flow in the previous iteration. If the sum of the absolute changes in flow for all links is less than a specified tolerance relative to the total system flow, the solution has been determined; otherwise the iteration is performed again.

2.4.3.2 Water Quality Capability

Rossman (2000) describes the water quality modelling capabilities of EPANET as follows:

1. Models the movement of non-reactive tracer material through the network over time;
2. Models the movement and fate of reactive material as it grows (disinfection byproducts) or decays (chlorine residual) with time;
3. Models the age of water throughout a network;
4. Tracks the percent of flow from a given node reaching all other nodes over time;
5. Models reactions both in the bulk water and at the pipe wall;

6. Uses n^{th} order kinetics to model reactions in the bulk water flow;
7. Uses zero or first order kinetics to model reactions at the pipe wall;
8. Accounts for mass transfer limitations when modelling pipe wall reactions;
9. Allows growth or decay reactions to proceed up to a limiting concentration;
10. Employs global reaction rate coefficients that can be modified on a pipe by pipe basis;
11. Allows wall reaction rate coefficients to be correlated to pipe roughness;
12. Allows time varying concentrations or mass inputs at any location in the network; and
13. Models storage tanks as being either complete mix, plug flow, or two compartment reactors;

These features give EPANET the capability to model the age of water (HRT) through the system, the decay of residual chlorine concentrations, and the formation of disinfection byproducts. A number of kinetic, transport and mixing equations are employed to calculate the water quality at a point in the system. A Lagrangian time-based transport algorithm is used to track the constituent being moved and mixed in the network. A detailed description of the reaction equations and transport algorithm can be found in Rossman (2000).

The water quality capabilities of EPANET differ from the biological models discussed in Section 2.4.2. The biological models have different competing kinetic reactions representing the interaction between water quality variables such as bacteria, organic matter and free chlorine. EPANET does not allow interaction between water quality variables. Additionally, the fate of these variables in EPANET are only represented by decay or saturation equations.

3.0 STUDY SITES

The contents of this chapter describe the two systems chosen for study during the project.

3.1 Taylorside/Ethelton Pipeline

3.1.1 Location

The Taylorside/Ethelton pipeline, shown in Figure 3.1, is part of the Melfort Rural Pipeline Association. This low pressure/ low flow distribution system was constructed in 1994 and installed below the frost line using a chain trenching technique. It is fed by the SaskWater Melfort/Kinistino regional treated water pipeline. Melfort is located approximately 250 km northeast of Saskatoon, Saskatchewan.

The Taylorside/Ethelton pipeline starts at Highway # 3 approximately 12 km northwest of Melfort, near the town of Beatty ('NW Connection' in Figure 3.1). Forty-two users are supplied treated water through this pipeline which terminates approximately 3 km west of Ethelton, Saskatchewan. The majority of the pipeline is west of Highway 368 and extends south of Highway 41. Three monitoring sites were established on the Taylorside/Ethelton pipeline. The monitoring sites where located at the Taylorside booster station ('Booster' in Figure 3.1), which provides the flow required for the water to reach the end users, a user site near the midpoint of the line ('Mid Point' in Figure 3.1) and a user site at the far end of the pipeline ('Far Point' in Figure 3.1). Individual users are denoted by a dot. Water entering the regional pipeline at the SaskWater treatment plant in Melfort was also monitored.

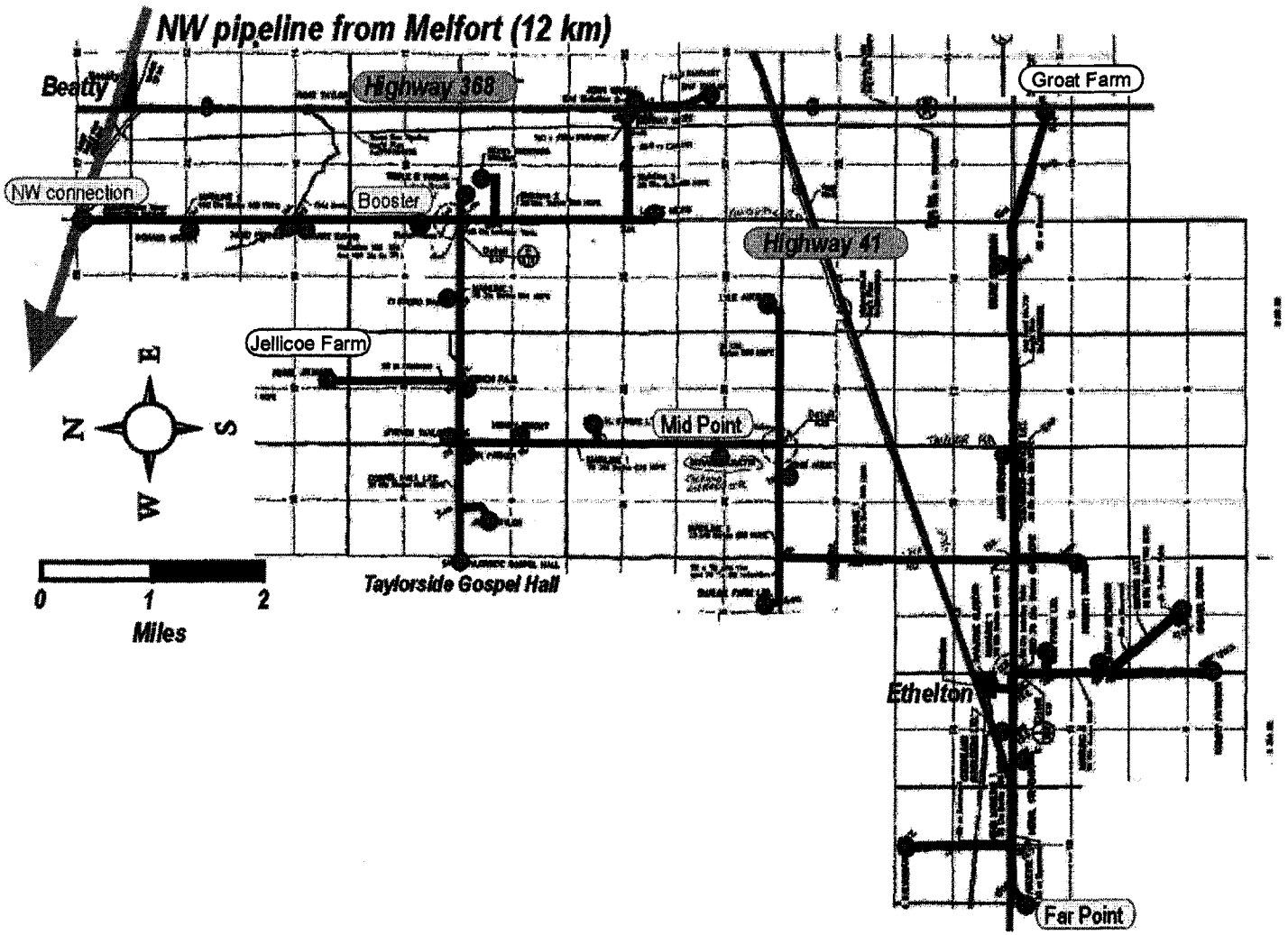

Figure 3.1 Taylorside/Ethelton Pipeline monitoring and sampling locations (after Putz and Mills, 2002).

3.1.2 Source and Treatment

Treated water is supplied to the Taylorside/Ethelton branch line via the regional pipeline that originates at the Melfort regional water treatment plant. The plant is situated just east of Melfort, Saskatchewan. Raw water is supplied to the Melfort plant via a pipeline from the Codette Reservoir on the Saskatchewan River. A conventional surface water treatment process (involving coagulation, clarification and filtration) is used to treat the water before it is chlorinated, stored and distributed to the regional pipeline. Pre-chlorination is implemented in addition to the other treatment steps.

3.1.3 Pipeline Characteristics

The pipeline is constructed of butt-fused high density polyethylene pipe. The main trunk of the branch system is approximately 22 km long and the pipeline serves an area of 181 km^2 (refer to Figure 3.1). Nominal pipe diameter ranges from 100 mm at the head of the line to 25 mm at the service connections. Most service connection lengths are in the order of several hundred meters, the longest being approximately 2400 m. The distribution system is a low pressure/ low flow type meant to provide only small flow rates over an extended period of time. Because the system is designed in this way, users are required to have an onsite storage and pumping system to supply household peak demands independent of the distribution system. The typical configuration includes a pressure sustaining valve to ensure upstream pressure does not fall below a set point, a flow meter for billing purposes, and a dual check or back-flow prevention valve. Flow into the cistern is controlled by a normally closed electric solenoid valve attached to a non-mercury filled float switch. The service connection into the house reduces from 25 mm to 19 mm before the pressure sustaining valve. The working volume of a cistern is typically 500 to 3000 litres, which is around 30 to 50 % of the total cistern volume. A device to restrict flow to below a maximum rate is frequently employed on low flow lines, but is not used here.

3.1.4 Users and Demand

The branch pipeline distributes water to 42 user sites for human consumption, livestock watering for smaller operations during the winter months, lawn and garden irrigation, spraying, and extensive livestock operations. During the study period, individual consumer demand varied from as little as 3.2 m^3/month up to as much as 1892 m^3/month.

3.2 Lucky Lake North Pipeline

3.2.1 Location

The Lucky Lake North Pipeline is shown in Figure 3.2. It is part of a much larger distribution system operated by the Coteau Hills Pipeline Association. Lucky Lake is approximately 160 km south of Saskatoon, Saskatchewan. Water is supplied to the branch pipeline from the Coteau Hills Mainline 1, a 150 mm HDPE pipeline which follows provincial Highway 42 to Lucky Lake. The head of the Lucky Lake North Pipeline is between booster stations 3 and 4 on the Mainline 1 Pipeline, approximately 3 km east of Lucky Lake. The pipeline was installed below the frost line in 1996 using a ploughing technique and is located between Highways 42, 45 and 646. Based on billing records, only seven users were supplied raw water during the study period, via this pipeline, which terminates approximately 5 km west of Birsay, Saskatchewan.

Monitoring equipment was set up at two user sites, one near the midpoint ('Mid Point' in Figure 3.2) and one at the far end of the pipeline ('Far Point' in Figure 3.2). Existing datalogging equipment was used to read flow and pressures at the pumphouses ('Booster #3' and 'Booster #4' in Figure 3.2).

3.2.2 Source and Treatment

Water for the Coteau Hills Pipeline is taken from a shallow bay on Lake Diefenbaker. Pressure and flow are provided by dedicated pumps in a large pumphouse constructed by SaskWater to provide irrigation water to the area. Lake water is pumped

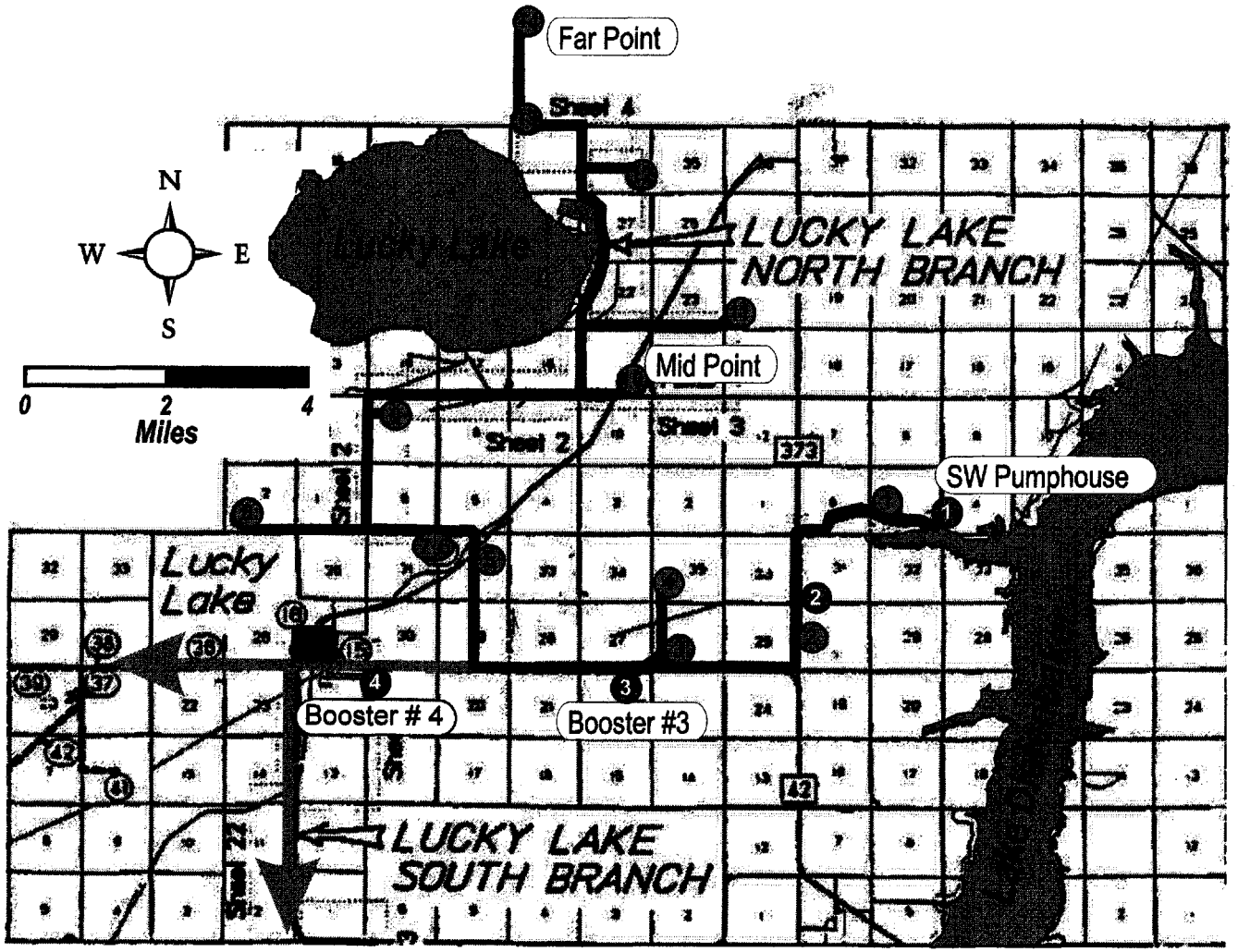

Figure 3.2 Lucky Lake North Pipeline monitoring and sampling locations. (after Putz and Mills, 2002).

directly into the distribution system without treatment. Users along the length of the line provide point of use treatment, generally in the form of staged cartridge filters followed by UV disinfection. One of the user sites monitored used a small multimedia filter to provide particulate removal in addition to the cartridge filters and UV sterilizer. Local runoff from the large area up-slope of the pumphouse typically causes elevated turbidity in the bay during spring melt and periods of rain. Contaminants from livestock and agricultural spraying are also present in Lake Diefenbaker (World Lakes Database, 2005).

3.2.3 Pipeline Characteristics

The pipeline is constructed of high density polyethylene pipe. The main trunk of the system is approximately 16 km long and the pipeline serves an area of 116 km^2. Diameter varies from 50 mm on the main trunk of the pipeline to 38 mm on individual services. Individual service lengths on the pipeline are, on average, longer than those observed on the Taylorside/Ethelton pipeline, the longest of which is approximately 1000 m. The system is designed for low pressure/low flow distribution requiring users to have storage and pumping systems similar to those on the Taylorside/Ethelton pipeline. However the flow rate into each of the cisterns is limited to 7.8 Lpm by a flow regulator.

3.2.4 Users and Demand

Users of the Lucky Lake North pipeline include agricultural and private livestock operations as well as two commercial pork farms. Irrigation and spraying water is not required from the distribution system as SaskWater has an extensive irrigation network in the area. The irrigation system is run only seasonally and as a result water from the distribution system is used to water wintering livestock at the few operations present. Seasonal impoundments, which are in some cases filled from the irrigation pipeline, are used to provide most water for lawn and garden irrigation. Consumer demand varied from as little as 8.7 m^3/month at one site up to 634 m^3/month at one of the two pork farms.

4.0 MATERIALS AND METHODS

This chapter describes the materials and methods used to collect the data for this research.

4.1 Overview

Hydraulic and water quality data were collected over the course of the study period (June 2000 to September 2001). Pressure and flow were monitored at five sites. For each site, data was collected at five minute intervals throughout the study period. The data set spans approximately 462 days and contains 133,000 time stamped flow and pressure pairs for each site monitored.

The Taylorside/Ethelton and Lucky Lake North pipelines were sampled for water quality parameters 24 and 23 times, respectively. The resulting water quality data set included 752 DOC measurements which resulted in 376 initial DOC readings and 376 BDOC values, 188 particle size analyses, 188 turbidity measurements, 188 temperature measurements, 200 epifluorescent bacteria counts, 96 residual chlorine measurements, and 32 heterotrophic plate counts. A total of 24 of the 32 HPC analyses were completed on the Taylorside/Ethelton pipeline during the period of expected peak activity (August 13, 2001 to September 18, 2001).

4.2 Flow and Pressure Data

Flow and pressure data were required to characterize system demand. Previously, little work had been done to characterize flow and pressure patterns in rural systems. The service connections at the homes contained a meter, pressure regulator and in some cases, a flow regulator. The existing instrumentation had insufficient

resolution for this study. Thus a monitoring apparatus was plumbed into the existing service connection to document user trends.

4.2.1 Equipment Used

Flow and pressure data was collected and stored on Campbell Scientific CR 10 X data loggers placed on site. Readings were collected by the data logger every second and the samples were averaged at the end of a 5 minute interval at which point the data was time and date stamped and stored in the memory of the data logger. The data was downloaded to a laptop computer with each site visit.

Flow and pressure were not monitored at the Taylorside/Ethelton pumphouse prior to this study. Flow was monitored through the use of an ultrasonic flow meter which could be installed without interrupting service. The flow meter transducers were placed on the outlet line from the pumphouse so that only the demand variation in the pipeline was monitored. A 4- 20 mA signal created by the meter head was monitored by the datalogger and converted to a flow rate. Figures 4.1, 4.2 and 4.3 show the flow meter, installation, and one of the pressure transducers. The Coteau Hills Booster Stations had monitoring equipment in place and it was used to collect flow and pressure readings for this study. Figure 4.4 shows the data collection equipment present in one of the Coteau Hills Booster Stations.

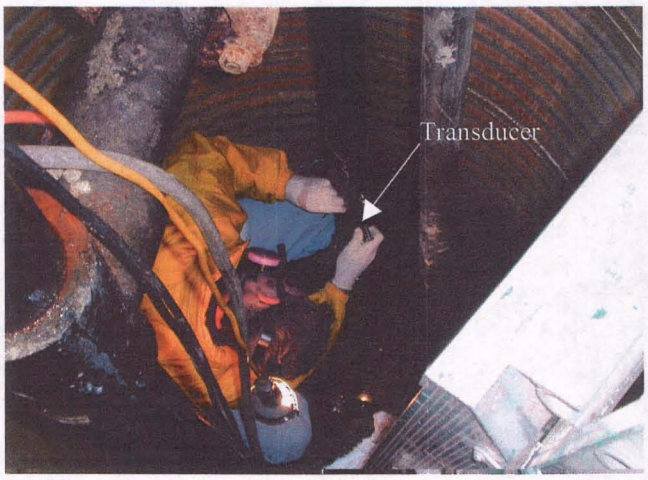

Figure 4.1 Installation of the Taylorside/Ethelton flow meter transducers.

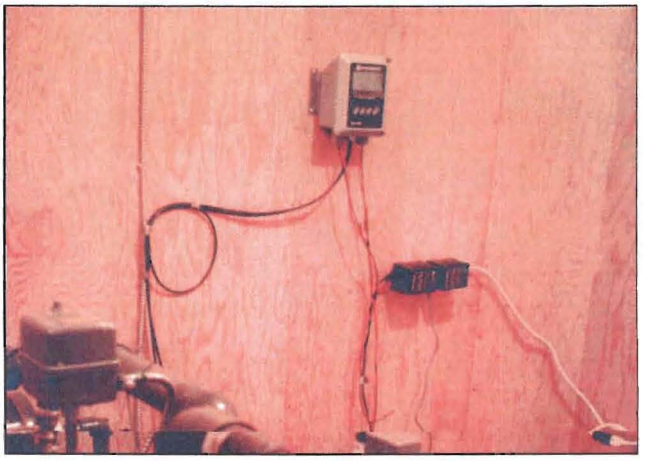

Figure 4.2 Taylorside/Ethelton ultrasonic flow meter readout/signal generator.

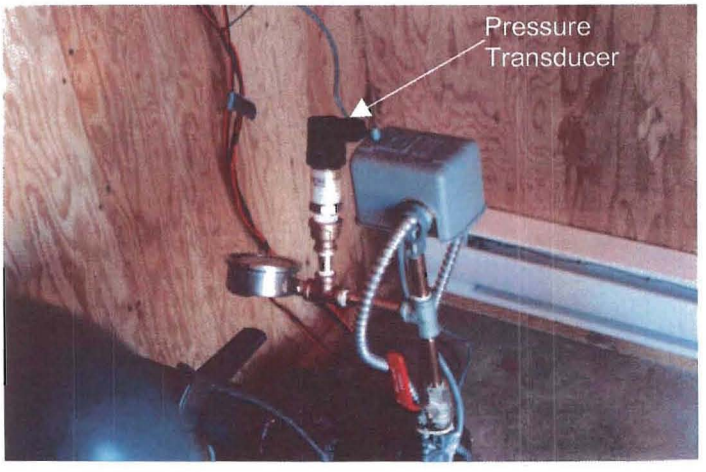

Figure 4.3 Taylorside/Ethelton inlet pressure transducer.

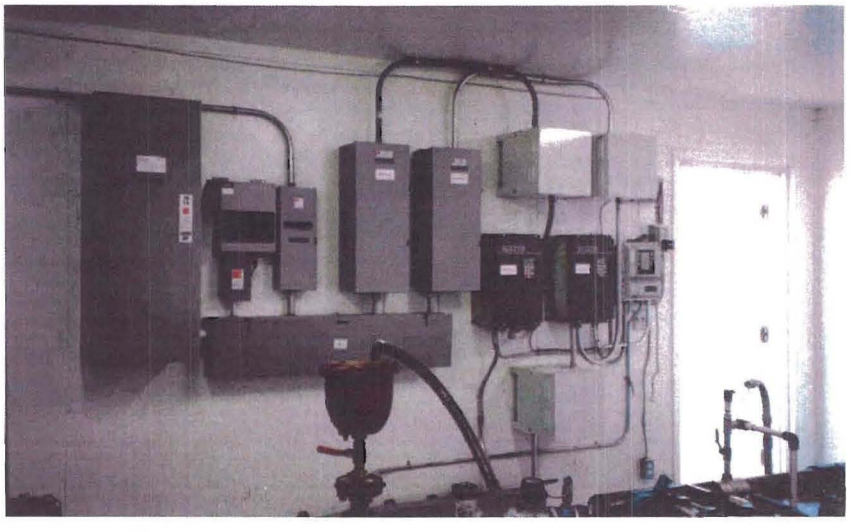

Figure 4.4 Coteau Hills Booster Station #3 electrical and datalogging equipment.

The user monitoring sites on the Taylorside/Ethelton line included Great Lakes Instruments online turbidity analyzers. The analyzers employed a dual light scattering method of measurement in which the turbidity was a function of the difference between the light from the source probe and that received by the receptor probe. The data logger was set to accept, average and store the 4-20 mA signal only when a flow signal from the meter was present.

The flow meters used at the user sites were 19 mm diameter, paddle wheel type. The meters did not have a display, only wiring for monitoring a pulse generated by each rotation of the paddle wheel. The number of pulses generated per unit volume was checked for each meter. The number of pulses per volume for each meter were entered into the data logger program to ensure accurate accounting of the flow. The data logger program calculated the volume based on the number of pulses observed in a 5 minute interval.

Line pressure was also monitored at the user sites. Pressure transmitters were used to generate a 4-20 mA signal corresponding to gauge pressure, and the data logger converted the signal into a pressure reading. The transmitters were checked using a hydraulic testing device. The multiplier used in the data logger program for converting the signal to a pressure was adjusted until the data logger output matched the pressure generated by the testing device at several different values within the transmitter range.

4.2.2 Configuration

An example of the site monitoring assemblies can be seen in Figure 4.5. These assemblies included a pressure transmitter at the front of the line, as close to the inlet pipe as possible and upstream of all other monitoring devices so that distribution system pressure would be recorded. When required, the turbidity meter was placed upstream of any flow control or flow metering devices to avoid the possibility of micro bubbles created by flow through the instruments being read as turbidity. The flow meter was placed last in line. At this location it was not expected to interfere with instruments upstream.

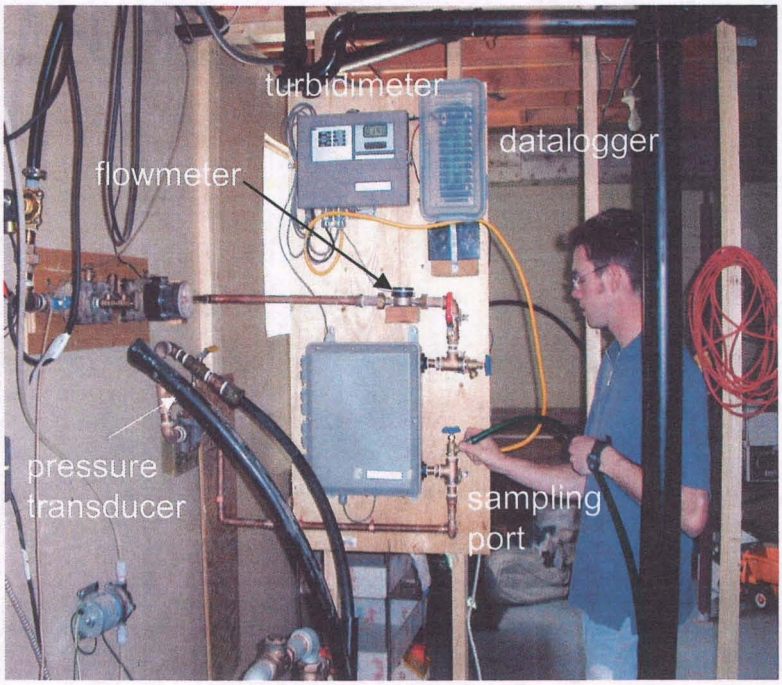

Figure 4.5 Typical user monitoring assembly and equipment.

The assemblies used for monitoring were pre-fabricated in the lab to avoid excessive interruption of service to the consumer and to make the installation and removal simple. The piping for the monitoring assemblies was constructed of rigid copper, brass and bronze threaded pipe and fittings. Connections were made using Teflon tape, a lead free thread lubricant/sealant certified for use on potable water joints and a lead free solder. In the case of the Taylorside/Ethelton sites, the connection of the monitoring assembly was as simple as splitting the service connection at the beginning of the control valve assembly and connecting copper lines to the feed and outlet pipes on the monitoring assembly. The assemblies for the Coteau Hills line were constructed mainly of threaded pipe and fittings and as such required more field fabrication, although most could be built and tested in the lab.

Water quality samples were taken from hose bibs installed on each of the monitoring assemblies. These sample points were installed upstream of the meters so that any volume required to flush the service line to the house would not be billed to the user or included in the demand patterns being recorded.

PC208W, version 2.2 from Campbell Scientific was used to program and interface with the dataloggers.

4.3 Water Quality Data Collection

The following describes the collection methods, laboratory preparation and testing protocol used to determine water quality parameters relating to the study.

4.3.1 Cleaning and Preparation of Containers

All sample bottles used for the collection of samples for BDOC, bacteria and particle counts were cleaned by filling them with a 1.5% (by volume) sulfuric acid solution, created using distilled de-ionized water and concentrated sulfuric acid. The containers were full of acid solution to remove any adhered material from previous samples. After a minimum of 48 hours contact with the acid solution, and immediately prior to the collection of the next set of samples, solution was emptied and the containers were rinsed with distilled water. Glassware used for storage, filtering, and incubation was cleaned using the same method.

4.3.2 Collection of Samples

For collection of samples on site, the service connection was flushed for 10 minutes to ensure that the water sampled was not influenced by the temperature gradient surrounding the home, and that the water sampled was from the branch line and not the service connection. New disposable latex gloves were worn at each site to avoid inadvertent contamination of the samples and protect the sampler from the chemicals used. The samples were kept in an ice packed cooler during transit, and stored at approximately 4°C in the Environmental Laboratory walk-in cooler until they were processed and analyzed.

4.3.2.1 BDOC

At each of the sites, two one litre Nalgene plastic sample bottles were filled to overflowing, capped, marked and kept in the cooler to inhibit any biological

consumption of BDOC before lab analysis. In the case of the Taylorside/Ethelton samples, 5 mL of sodium thiosulphate was also added prior to sampling to destroy the chlorine residual and avoid further reduction of organic matter by chlorine as recommended by Standard Method 9060A (APHA, 1992).

4.3.2.2 Bacteria

A 20 mL sample of water was taken from each site and placed in a dedicated glass container with a Teflon lined lid for epifluorescent bacterial counts. The pipette used to measure the 20 mL was rinsed several times with sample water before adding the water to the sample container. Bacteria in the samples were preserved with 1 mL of formaldehyde solution (37%) which had been filtered in the lab immediately prior to the site visit. The Taylorside/Ethelton samples were also fixed with 1ml of sodium thiosulphate to stop the reduction of bacteria by the chlorine residual.

4.3.3 Testing Performed On Site

Samples for parameters which could not be fixed, or would be affected by transit had to be measured on site to ensure that accurate in-situ conditions were recorded. Figure 4.6 shows the field kit used to collect these parameters.

4.3.3.1 Chlorine

Free and Total Chlorine were tested using Standard Method 4500-CI-F (APHA, 1992).

4.3.3.2 Turbidity

Turbidity was measured on site using a Hach 2100 turbidimeter using Standard Method 2130 B (APHA, 1992).

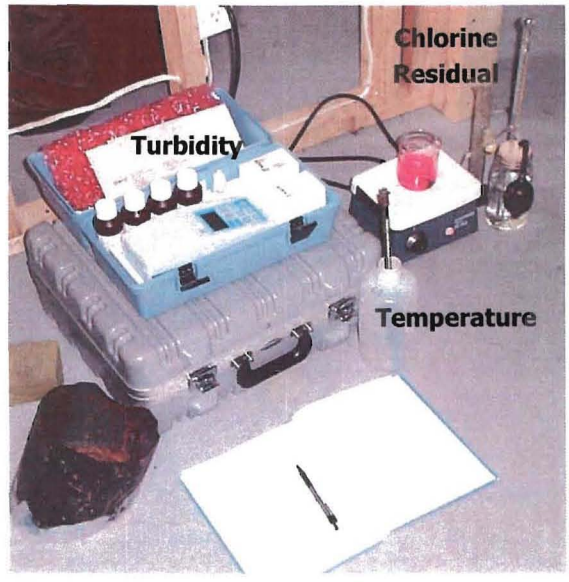

Figure 4.6 Field kit for collection of chlorine residual, turbidity and temperature.

4.3.3.3 Temperature

The temperature of the sampled water at each site was taken using a thermometer and the value recorded after the reading stabilized. The thermometer was rinsed several times with distilled water before being immersed in the sample.

4.3.4 Laboratory Testing

The following sub-sections outline the testing methods used for laboratory preparation and analysis of the parameters not measured on site.

4.3.4.1 Particle Counting

A 1000 mL sample was collected from each of the study sites and analyzed using a MetOne WGS267 particle counter. Particles in the 2-5, 5-10, 10-15, 15-20, 20-40 and greater than 40 micron ranges were measured and recorded.

4.3.4.2 BDOC

The concentration of BDOC was determined using the revised Billen-Servais Method outlined in Servais et al. (1989). This method is described in Section 2.1.1.3.

The source of heterotrophic bacteria for inoculation and incubation was South Saskatchewan River raw water.

Dissolved organic carbon analyses were performed by the Environment Canada Lab at NWRI in Saskatoon using a persulphate/UV oxidation technique with a reported detection limit of 0.1 mg/L. Two sets of initial and final samples for each site were prepared and analyzed using the procedures described below. Blanks were also prepared in duplicate in the same way using ultra-pure water (from Environment Canada) and submitted with the site samples. Initially, distilled water had been used for blank preparation. High values of DOC were noted in the blanks made using distilled water so ultra-pure water was then substituted for the distilled.

4.3.4.2.1 Sterilizing

To remove the suspended organics and sterilize the sample water without destruction of DOC, samples were filtered in the lab using vacuum filtration through a 0.2 micron polycarbonate filter. The filters were rinsed with distilled de-ionized water prior to sample filtration to remove any dissolvable contaminants from the filter that may adversely affect the outcome of the analysis. A 500 mL sample from each of the two bottles collected at each site were filtered and split between two glass incubation bottles that had Teflon lined lids. One of the split samples would provide the initial DOC for the site and the other the final. Two pairs of blank samples (four in total) were also prepared in this manner and split into the initial and final bottles as noted above.

4.3.4.2.2 Inoculum

The basis of the BDOC method used is that natural raw water would contain an adequate cross section of bacterial species that would consume all of the available biodegradable organic matter.

A sample of river water was taken and filtered in the lab to remove any suspended organic matter and protozoa. Theoretically, only heterotrophic bacteria would remain. Samples of the inoculum water were analyzed periodically using epifluorescent techniques to ensure an adequate number of organisms were present in the water.

The filtered water was added to the DOC bottles in the ratio of 1 mL inoculum to 100 mL of sample water. Both the initial and final samples were inoculated with only one exception. One of the blank samples from each pair of blanks was inoculated while the other was not. This was done to determine the effect of the inoculum on the DOC readings at the initial and final and final stages as well as note any changes in the non-inoculated ultra-pure water samples which might be indicative of contamination from the container.

4.3.4.2.3 Incubation

For one set of the split samples, 100 mL of the prepared sample was bottled in sterilized nalgene containers from the Environment Canada lab. These samples were returned to the Environment Canada lab in order to establish initial DOC levels. The final samples were incubated in the dark for 28 days at 20°C to allow consumption of the available BDOC by the bacteria in the inoculum. At the end of the 28 days the samples were removed from the incubator and 100 ml of sample was again sent to the Environment Canada lab for analysis.

4.3.4.3 Epifluorescent Bacteria Counts

Samples from each site were taken to determine the number of organisms present in the water. The Epifluorescent method used does not differentiate between live and dead cells, viable or inactive. The counts were of the total number of organisms in the water. 4'6-diamidino-2-phenylindole (DAPI) was used to stain the DNA of the cells. Under the correct wavelength of light, the cells appear violet to blue in color. Instruction on the method and access to the UV microscope used to fluoresce and count the bacteria was provided by Environment Canada.

The Epifluorescent method involves dilution of the samples with Autoclaved water and addition of the DAPI stain. As DAPI is light sensitive, the majority of this work must be done in extremely low light. A period of mixing and reaction is required after which the samples are each filtered through separate 0.2 μm black polycarbonate filters which retains the cells. The filters are placed on slides and placed under the

microscope. The view at 100 times magnification has a 10 by 10 micron grid. The bacteria within the 10 by 10 grid are counted, recorded, and the view is moved up and to the right. 10 grids are counted in this manner and the view is moved left approximately 10 grid widths and the procedure is repeated going down and to the right. 20 grids are counted or 200 bacteria in total are counted, which ever occurs first. The values are entered into a spread sheet developed by Environment Canada and the total number of organisms per mL is calculated.

4.3.4.4 HPC

Samples for heterotrophic plate counts were collected from the Taylorside/ Ethelton pipeline to determine the viable fraction of the organisms present in the water. Samples were collected using bottles, reagents and procedures provided by Saskatchewan Health specifically for HPC analyses. The samples were forwarded to the Provincial lab for enumeration. The general reasoning used in this test is that each of the colonies formed on the plate originate from a single cell. The number of colonies after incubation are counted and the number of Colony Forming Units (CFU) per volume of water (typically 100 mL) are calculated.

4.3.5 Biofilm Sampling Program

In 2001, during what was believed to be the peak period of activity, pieces of the distribution main were removed to investigate if biofilm had formed in the pipe. The locations chosen to sample were based on low usage and long service length, which results in long residence times. Locations with these characteristics were expected to be the most susceptible and thus the most likely to have established biofilms. Two sites were chosen on each of the pipelines. Users were given advance notice of when the sampling would occur so they could ensure the cistern had adequate water. The pipes sampled were taken from individual service lines so only one user experienced interruption of service at a time.

4.3.5.1 Pipe Sampling Procedure

Figure 4.7 shows some of the pipe sampling procedure for the biofilm analysis. Service lines were exposed by excavation of the cover soil. The section of pipe was partially cleaned of mud and debris and the pipe was pinched off using "pipe squeezers" to stop flow. A 1 m section of pipe was removed from the line using plastic pipe cutters. The pipe sample was brought out of the trench and the exterior of the pipe was cleaned first with water to remove remaining mud and soil, then sterilized with a 10% ethanol solution. Latex gloves were worn and sterilized with the ethanol solution. The 300 mm on each end of the 1 m sample were cut off and discarded. The remaining pipe was cut into 150 mm sections and each placed in a 1 L Nalgene bottle which had been cleaned prior to use. The bottles were then filled with ultra-pure water and capped. The samples were transported back to the Environmental Engineering lab in a cooler.

Figure 4.7 Pipe sample preparation during biofilm investigation.

4.3.5.2 Sonication

Sonication was used to remove the bacteria from the pipe surface as it is a non-intrusive method which does not affect the viability of the bacteria (Mathieu et al., 1993). Equipment used in the sonication of the samples was rinsed with 2% sulfuric acid

solution followed by distilled de-ionized water between samples. The sonication probe was inserted into the sample bottle containing the pipe and the water was sonicated at 90% of the maximum frequency for 10 minutes. The interior of the pipe was also scoured with a new, clean bottle brush inside the sample bottle. The brush was cleaned with sulfuric acid and distilled de-ionized water between samples. The pipe sample was removed from the bottle and the surface rinsed with distilled de-ionized water such that the rinse water ran into the sample bottle.

4.3.5.3 Bacterial ID/Enumeration

Samples for HPC (50 mL), epifluorescent counts (50 mL) and species identification (50 mL) were taken from the sonicated sample water. HPC analyses were performed in the Environmental Engineering Lab using Standard Method 9215 D (APHA, 1992). The epifluorescent counts were performed as discussed above and identification of the bacterial species was performed by Norwest Labs in Calgary, AB. The volume of the remaining water was measured and recorded so that the total number of organisms present in the water collected (presumably from the surface of the pipe) could be calculated once the per mL concentrations were known. The interior diameter and length of the pipe sample were measured so that the surface area could be calculated. The biofilm concentrations were then calculated by taking the total number of organisms divided by the interior surface area of the pipe.

5.0 HYDRAULIC DATA, ANALYSIS AND DISCUSSION

This chapter discusses and analyzes the hydraulic data collected over the course of study. Representative examples of the hydraulic data collected at the two pipeline sites are presented. The entire data set is attached in a data CD. A listing of the data set files is presented in Appendix A.

5.1 Taylorside/Ethelton Pipeline

The Taylorside/Ethelton pipeline was continuously monitored from mid-June 2000 to the end of September 2001. Dataloggers located at the Taylorside/Ethelton Booster Station, Midpoint and Farpoint locations were used to collect time averaged flow and pressure readings at five minute intervals. Figure 5.1 shows the flow rate, inlet pressure head and discharge pressure head at the Taylorside/Ethelton pumphouse as daily averages, over the study period. Table 5.1 summarizes the average values of flow and pressure head. In August of 2001, the inlet and discharge pressures at the Booster Station were increased. The Melfort water treatment plant records do not indicate why this change occurred. The 2001 values for pressure head, shown in Table 5.1, are average values before and after the increase.

Figure 5.2 gives an example of hourly flow and discharge pressure head variation for a typical week in 2000. Figure 5.3 shows an example of a week with a high hourly demand. The "typical week" was chosen based on the occurrence of a day during the week exhibiting the average daily flow rate observed for the period. The "week with high hourly demand" week was chosen based on the occurrence of a day during the week exhibiting the peak hourly flow rate observed for the period.

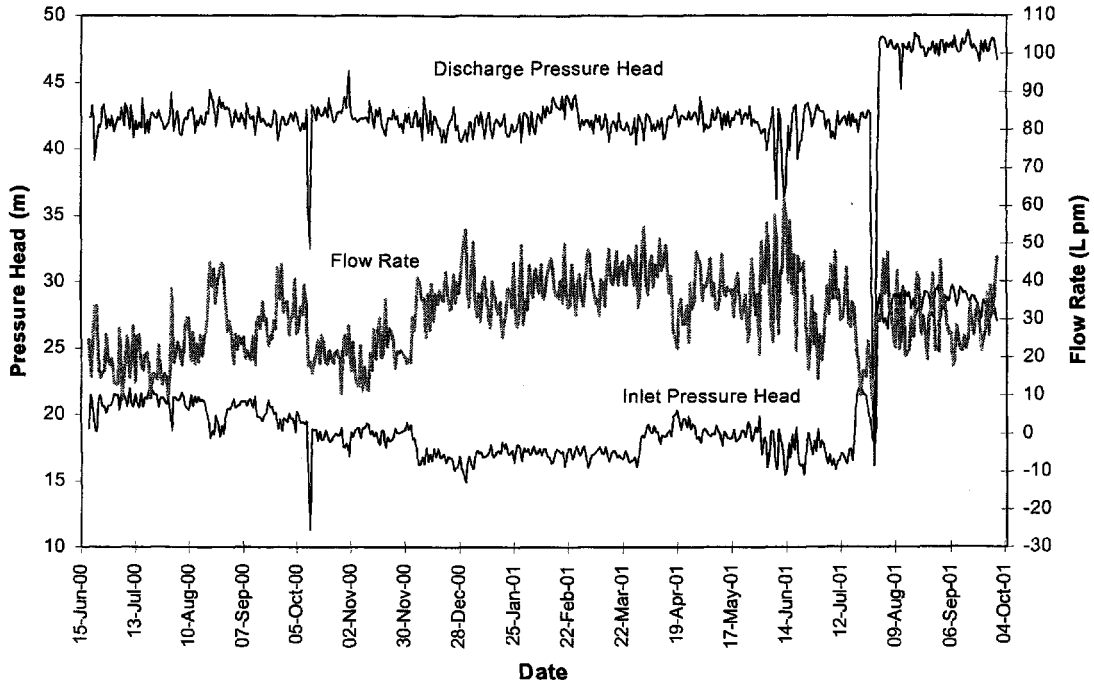

Figure 5.1 Daily average flow and pressure head recorded at the Taylorside/Ethelton Booster Station from mid-June 2000 to October 2001.

Table 5.1 Taylorside/Ethelton Booster Station characteristic daily average values for the period of study.

Parameter	2000 (June 22 - Dec. 31)	2001 (Jan. 01 – Sept. 30)
Average Flow (Lpm)	26 23.31[2]	35 28.31[2]
Peak Flow (Lpm)	54	62
Average Inlet Pressure Head (m)	19 4.48[2]	17.8 / 28[1] 4.39 / 5.07[2]
Average Discharge Pressure Head (m)	42 4.24[2]	42 / 47.5[1] 5.82 / 3.38[2]

[1] Pressure heads shown are average values before and after inlet and discharge pressures increased in August 2001.
[2] Standard deviation.

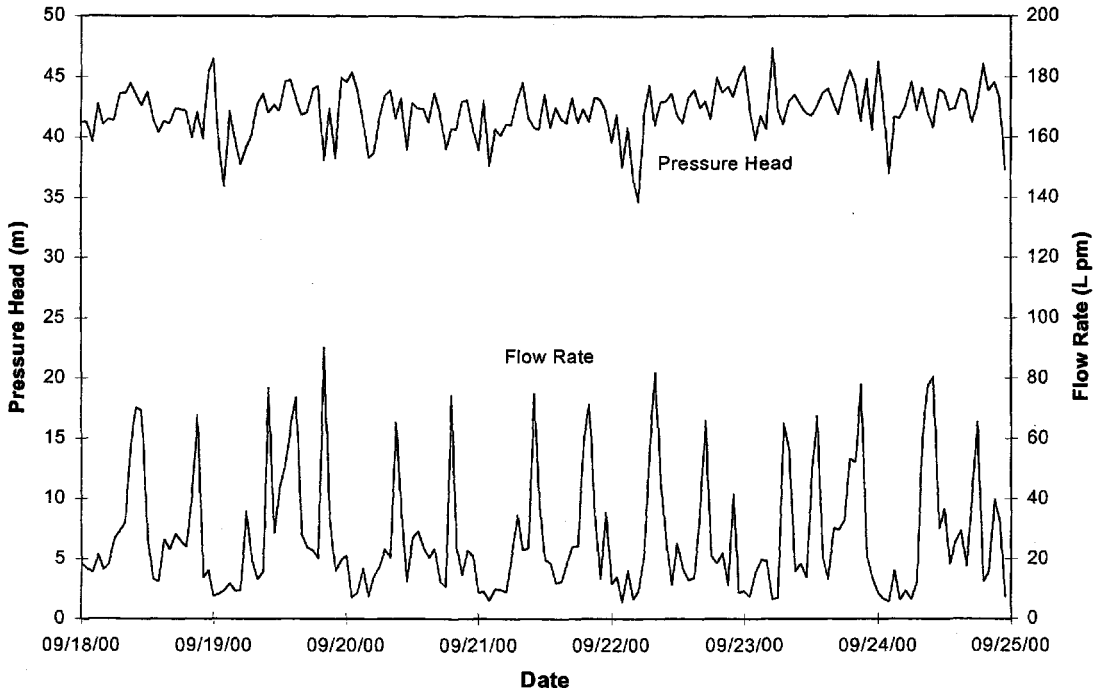

Figure 5.2 Example of hourly flow rate and discharge pressure head recorded at the Taylorside/Ethelton Booster Station for a typical week in 2000.

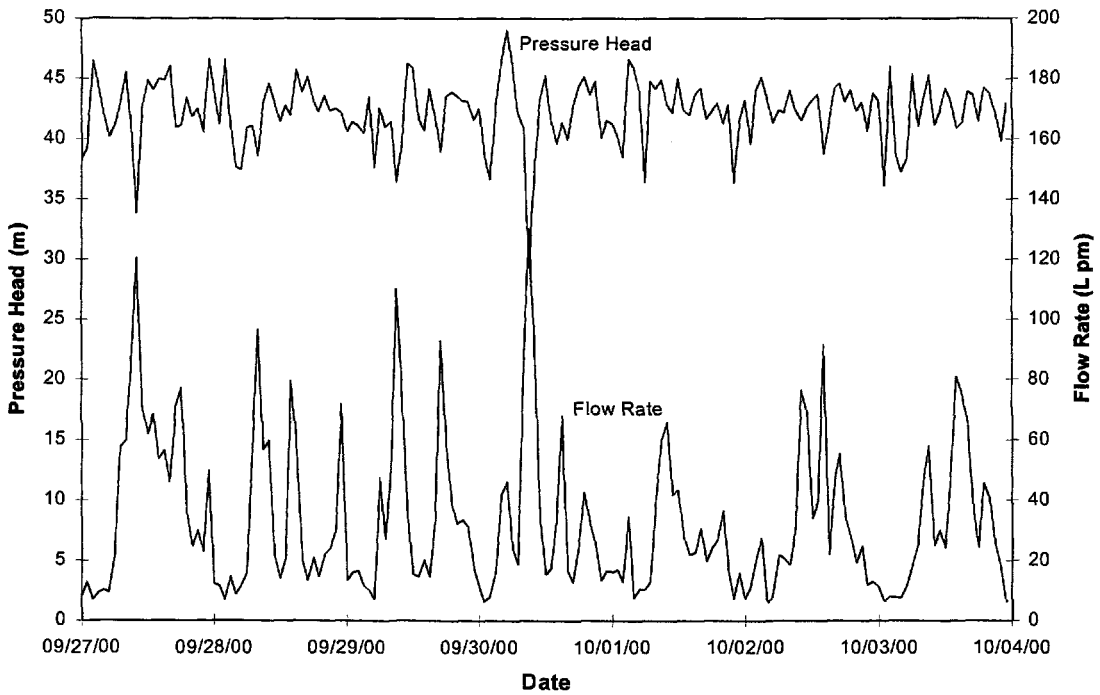

Figure 5.3 Example of hourly flow rate and discharge pressure head recorded at the Taylorside/Ethelton Booster Station for a week with high hourly demand in 2000.

Figures 5.4 and 5.5 show examples of flow and pressure head variation for a typical week and a week with high hourly demand in 2001. Table 5.2 summarizes average, minimum and maximum values of pressure head and flow for the periods shown in Figures 5.2, 5.3, 5.4 and 5.5. The peak flow rates observed in the 3rd quarter of 2001 were higher than in the 3rd quarter of 2000 and thus the average pressure head values were lower. The pressure head values observed also appeared to be more variable in the 3rd quarter of 2001 with a standard deviation in pressure head values of 7.2 m, prior to the increase in pressure, as opposed to 2.5 m for the same quarter of 2000.

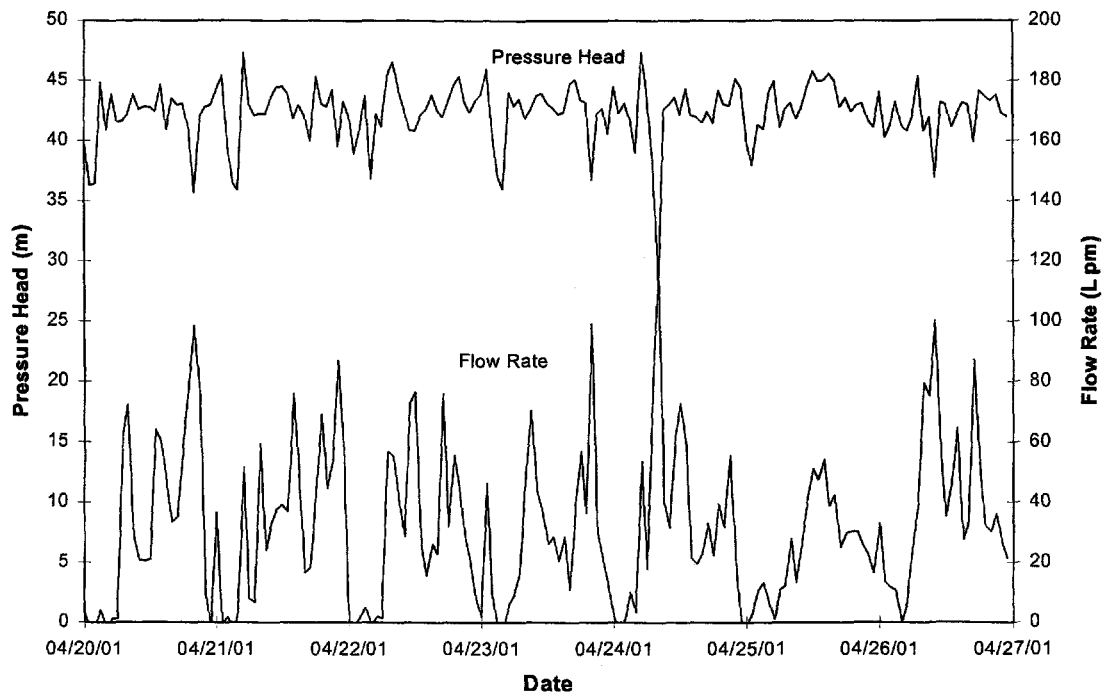

Figure 5.4 Example of hourly flow rate and discharge pressure head recorded at the Taylorside/Ethelton Booster Station for a typical week in 2001.

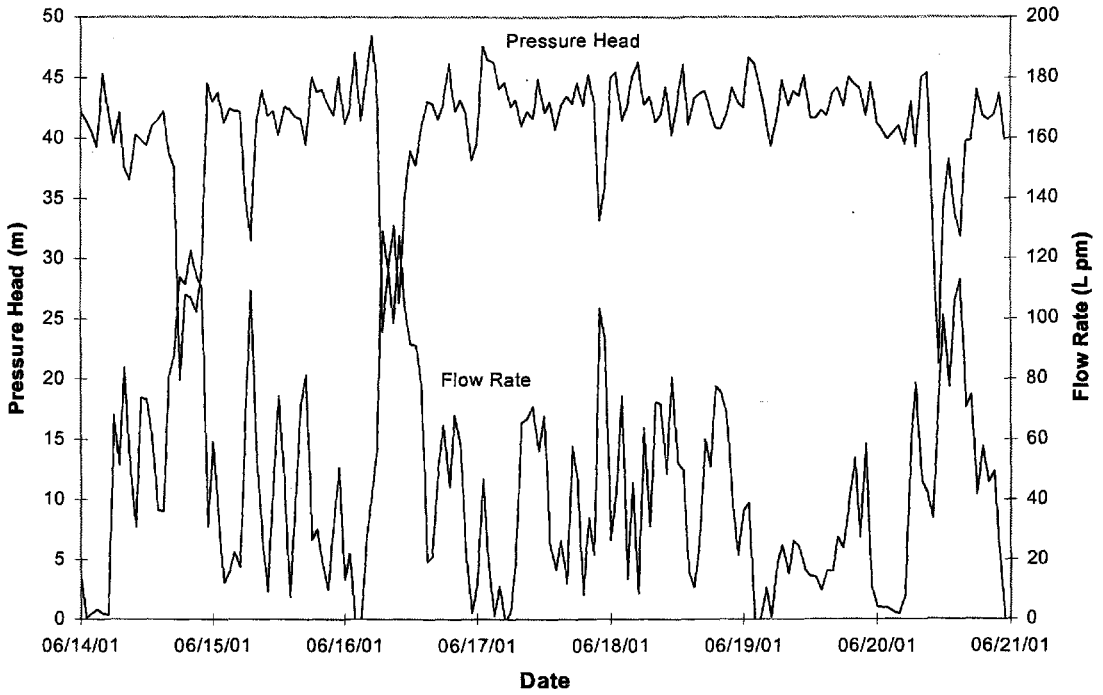

Figure 5.5 Example of hourly flow and discharge pressure head recorded at the Taylorside/Ethelton Booster Station for a week with high hourly demand in 2001.

Table 5.2 Summary of flow and pressure head values for a typical week and a week with high hourly demand in 2000 and 2001 for the Taylorside/Ethelton Booster Station.

Period	Avg. Hourly Flow (Lpm)	Min. Hourly Flow (Lpm)	Max. Hourly Flow (Lpm)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Typical Week[1] – 2000 (Sept. 18 - Sept. 25, 2000)	27 23.2[2]	5	90	42 2.42[2]	35	47
Week with High Hourly Demand[1] – 2000 (Sept. 27 - Oct. 4, 2000)	33 25.9[2]	6	130	42 2.66[2]	29	49
Typical Week – 2001 (Apr. 20 - Apr. 27, 2001)	32 25.5[2]	0	117	42 2.39[2]	28	47
Week with High Hourly Demand – 2001 (June 14 - June 21, 2001)	44 33.2[2]	0	129	41 4.72[2]	29	49

[1] Typical week and week with high hourly demand defined in section 5.1.
[2] Standard deviation.

Flow and pressure were recorded concurrently at the Midpoint and Farpoint sites. Figures 5.6 and 5.7 show the pressure head and flow rate variations into the user's cisterns at these locations for a typical week in 2000. Pressure head values are averaged over an hour. The flow rate is shown as five minute intervals in order to provide adequate resolution. The difference in pressure head values between the two user sites are a result of an elevation increase to the Farpoint and frictional losses in mains that decrease in diameter towards the system periphery. The data contained in Figures 5.6 to 5.13 is quantified in Tables 5.3 and 5.4, which summarize the average cistern fill rate, consumption, and pressure head variations observed at the Midpoint and Farpoint sites.

The effects of a period of high system flow in 2000 are shown for the Midpoint and Farpoint in Figures 5.8 and 5.9. Under increased system demand, the average pressure head at these two sites remained relatively unchanged. However, the range of pressure head values increased at higher system flow rates. Due to increased demand, the number of solenoid valve actuations increased from 4 to 5 at the Midpoint user. The average filling rate for the Midpoint site remained unchanged but an increase was noted at the Farpoint.

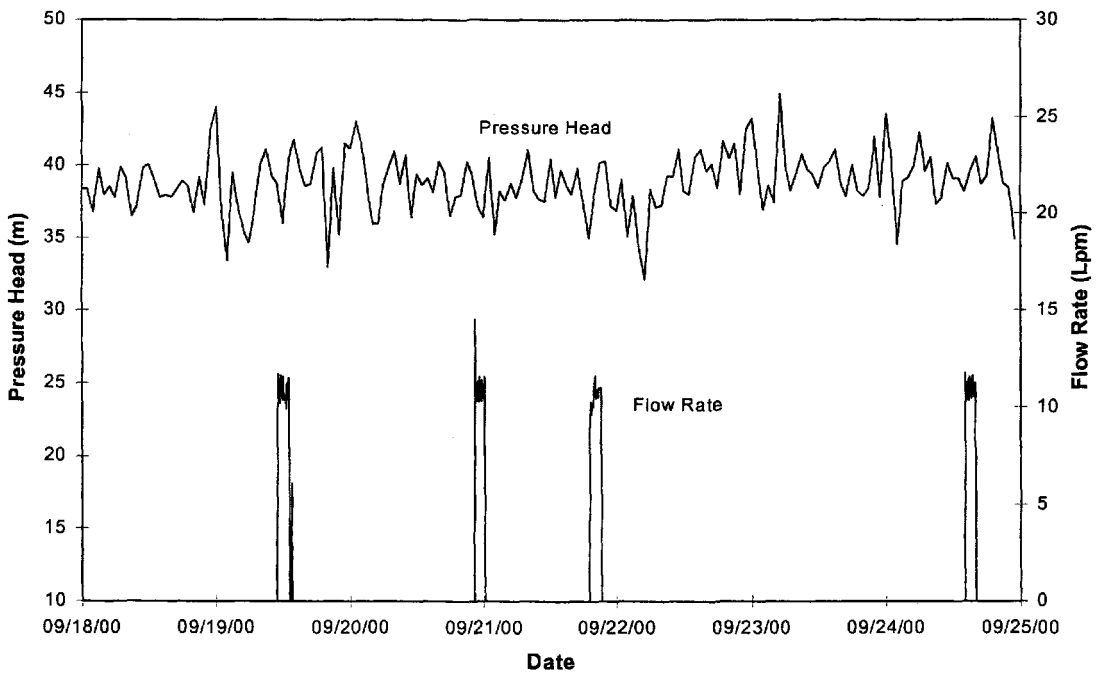

Figure 5.6 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Midpoint for a typical week in 2000.

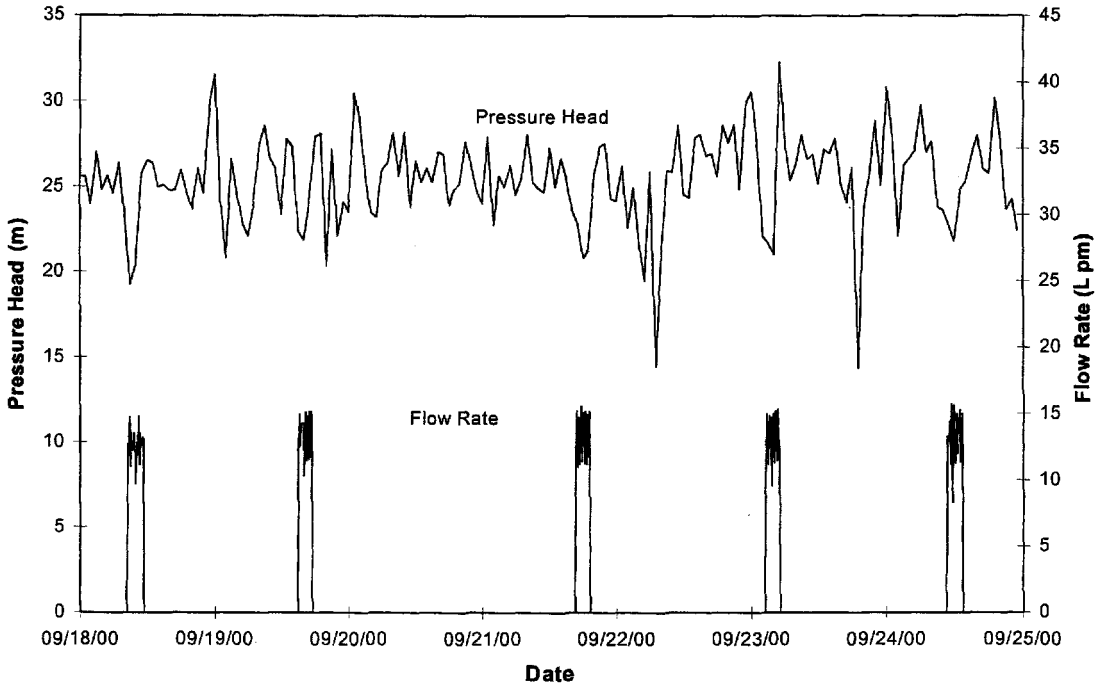

Figure 5.7 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Farpoint user for a typical week in 2000.

Table 5.3 Summary of cistern fill rate, consumption and pressure head values for a typical week and a week with high hourly demand in 2000 and 2001 for the Taylorside/Ethelton Midpoint site.

Period	Fill Rate (Lpm)	Consumption (L)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Typical Week[1] – 2000 (Sept. 18 to Sept. 25, 2000)	11	5,600	39 3.14[2]	28	44
Week with High Hourly Demand[1] – 2000 (Sept. 27 to Oct. 4, 2000)	11	6,700	39 2.72[2]	26	47
Typical Week – 2001 (Apr. 20 to Apr. 27, 2001)	11	8,700	38 2.97[2]	23	45
Week with High Hourly Demand – 2001 (June 14 – June 21, 2001)	11	6,500	36 5.94[2]	13	45

[1] Typical week and week with high hourly demand defined in section 5.1.
[2] Standard deviation.

Table 5.4 Summary of cistern fill rate, consumption and pressure head values for a typical week and a week with high hourly demand in 2000 and 2001 for the Taylorside/Ethelton Farpoint site.

Period	Fill Rate (Lpm)	Consumption (L)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Typical Week[1] – 2000 (Sept. 18 to Sept. 25, 2000)	13	10,600	25 3.51[2]	14	32
Week with High Hourly Demand[1] – 2000 (Sept. 27 to Oct. 4, 2000)	14	11,700	26 3.35[2]	11	34
Typical Week – 2001 (Apr. 20 to Apr. 27, 2001)	14	56,000	23 4.20[2]	6	32
Week with High Hourly Demand – 2001 (June 14 – June 21, 2001)	13	16,300	23 6.36[2]	1	33

[1] Typical week and week with high hourly demand defined in section 5.1.
[2] Standard deviation.

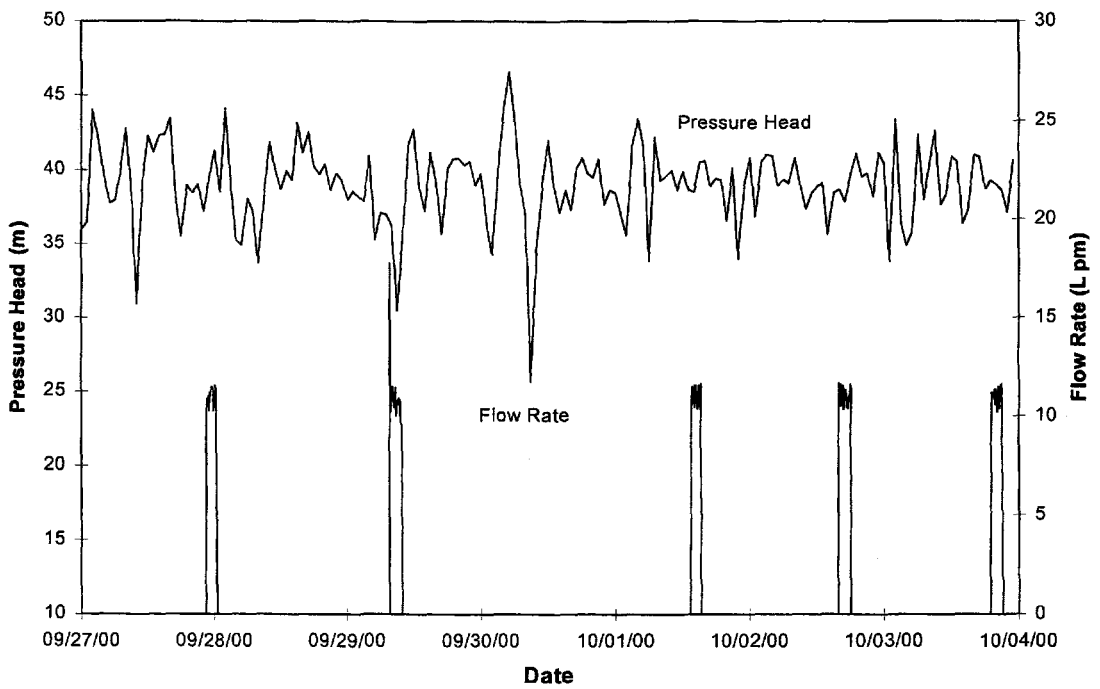

Figure 5.8 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Midpoint for a week with high hourly demand in 2000.

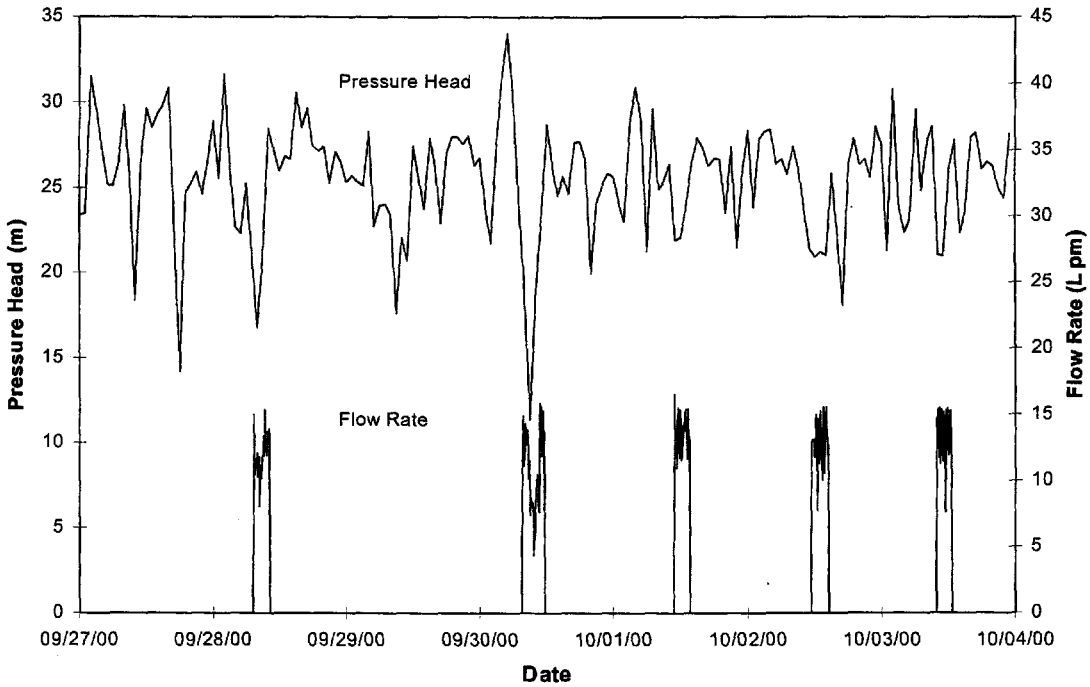

Figure 5.9 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Farpoint for a week with high hourly demand in 2000.

Monitoring over the winter months identified a trend very different from urban water supply systems. In this rural distribution system, the flow rates increased during the winter months. Cattle were wintered at the Farpoint site, causing the site to become a large consumer of water. Figures 5.10 and 5.11 give examples of the pressure head values and flow patterns at the Midpoint and Farpoint sites in spring 2001. The Farpoint site was still wintering cattle during the period shown in Figure 5.11. As a result, the number of solenoid actuations at the Farpoint site was greatly increased and the duration of filling approached 12 hours on several occasions. The Midpoint site does not winter livestock and had lower demand than the Farpoint site over the same period.

Figures 5.12 and 5.13 illustrate the pressure head variation and flow rate at the user sites during a period of increased system flow later in 2001 as was evident by observations at the Booster Station. Consumption at the two user sites was lower than that observed prior to this event, suggesting the increased demand was caused by other users in the network. The pressure head values were highly variable during this period.

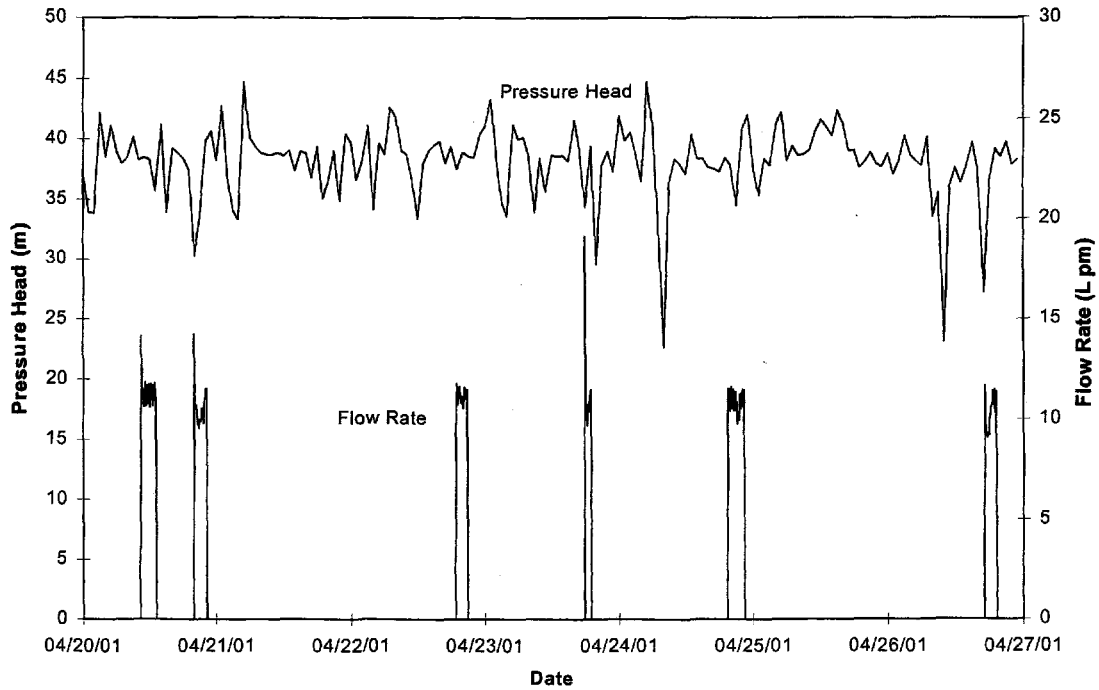

Figure 5.10 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Midpoint for a typical week in 2001.

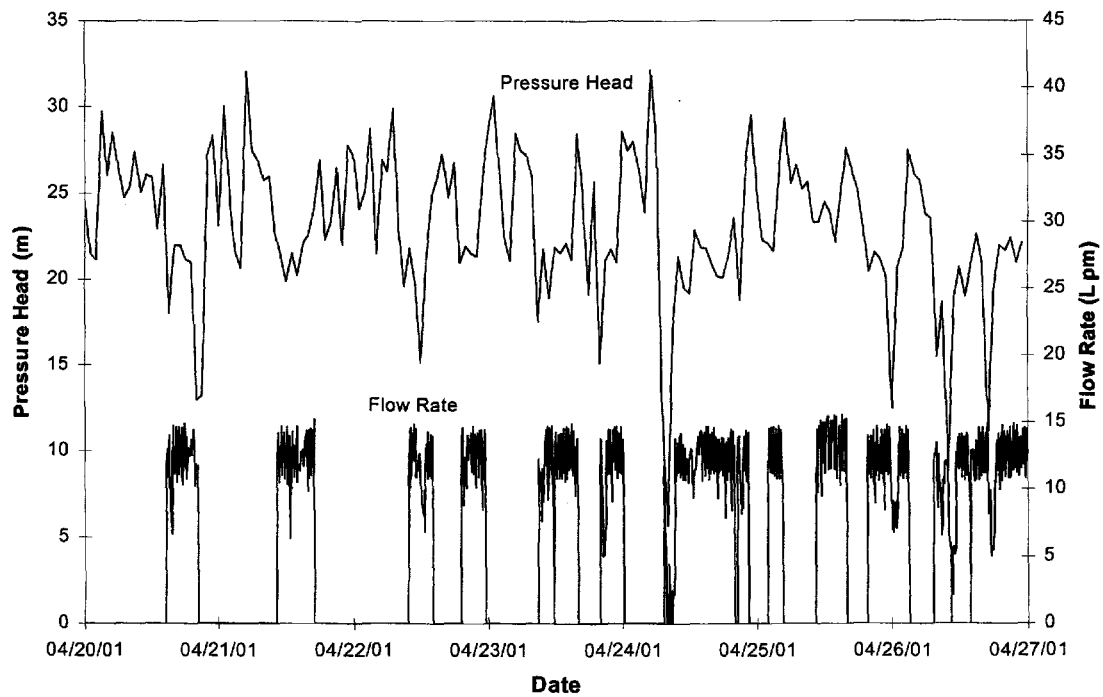

Figure 5.11 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Farpoint for a typical week in 2001.

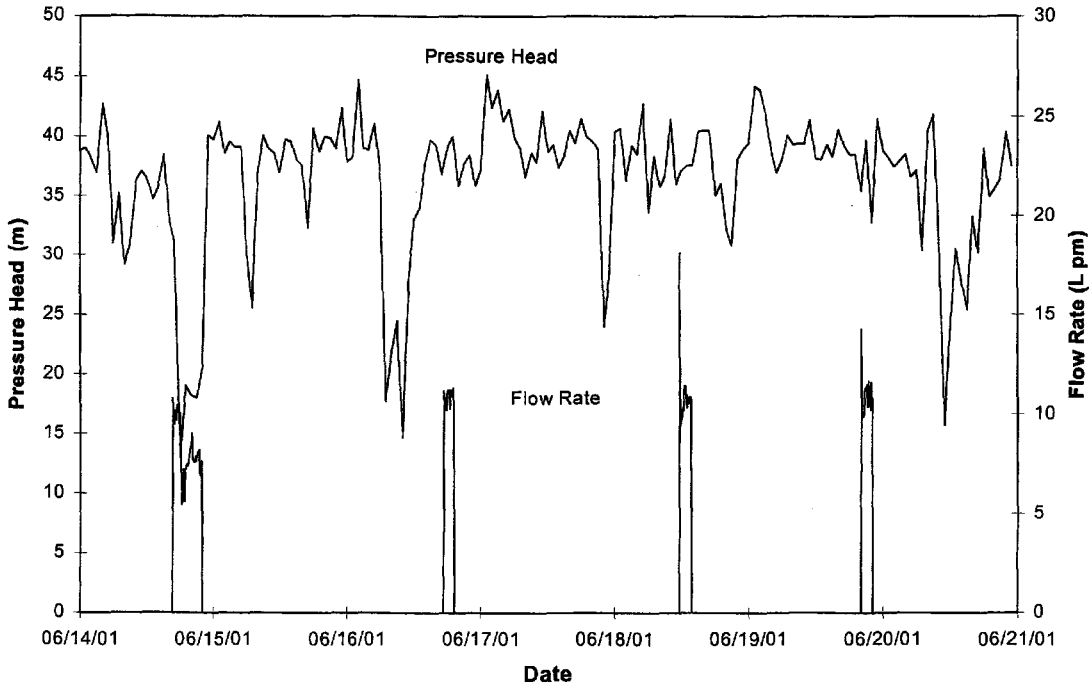

Figure 5.12 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Midpoint for a week with high hourly demand in 2001.

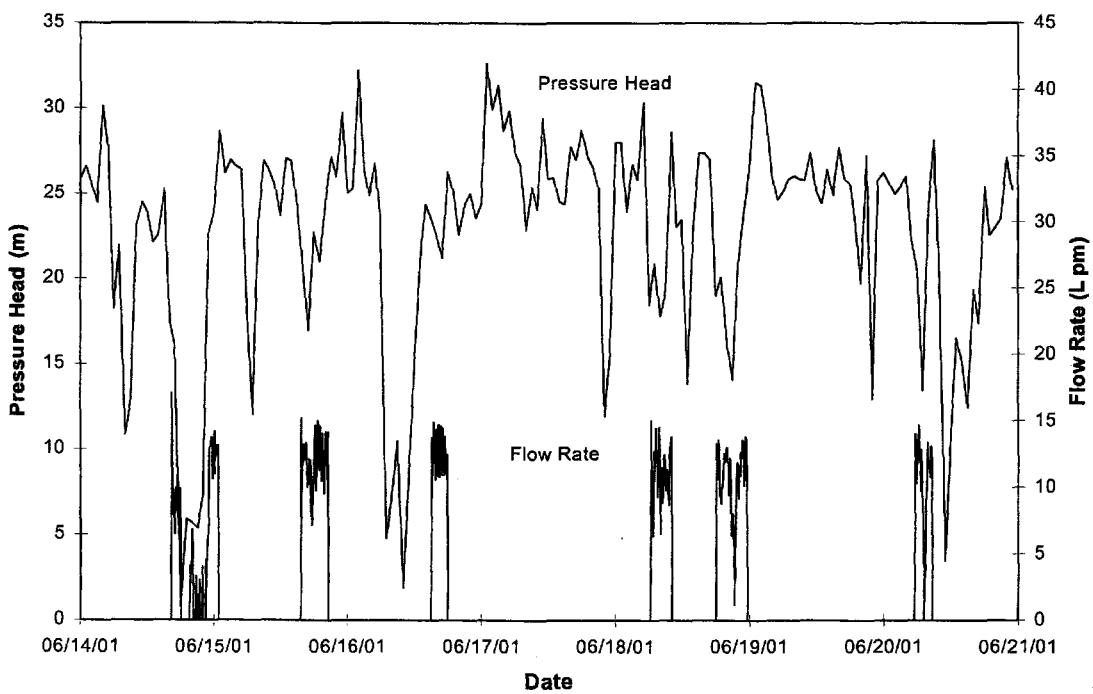

Figure 5.13 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Farpoint for a week with high hourly demand in 2001.

Periods of low pressure in the system continued until August of 2001 when the pump set point pressures for both the regional line and the Taylorside/Ethelton line were increased. Figures 5.14, 5.15 and 5.16 show the increased pressure in the Taylorside/ Ethelton pipeline. Table 5.5 summarizes the flow and pressure head for each of the monitoring sites for the period following the increase in pressure. The increased pressure appears to have a greater effect on residual pressures in the portions of the network near the Booster Station and Midpoint. However, pressure was still widely variable and fell below the acceptable operating level of 140 kPa (14.1 m pressure head) recommended by the Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers (1997) at the periphery of the network (Farpoint). This minimum recommended pressure has been adopted and is widely used in the municipal design industry.

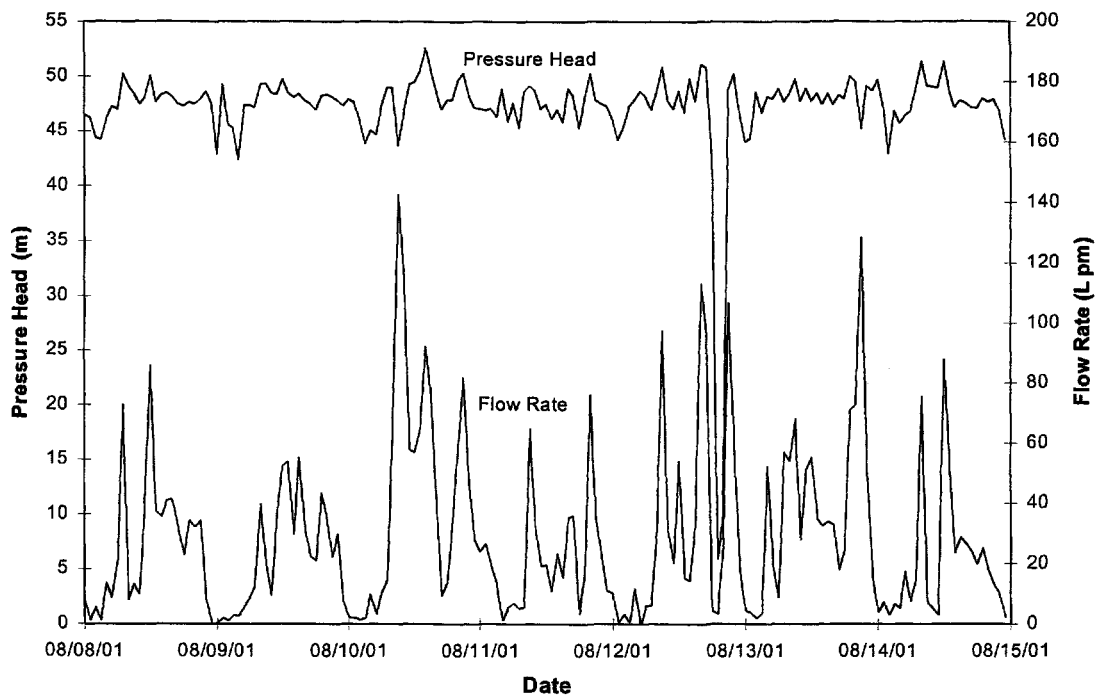

Figure 5.14 Example of hourly flow rate and pressure head recorded at the Taylorside/ Ethelton Booster Station from August 8 to 15, 2001.

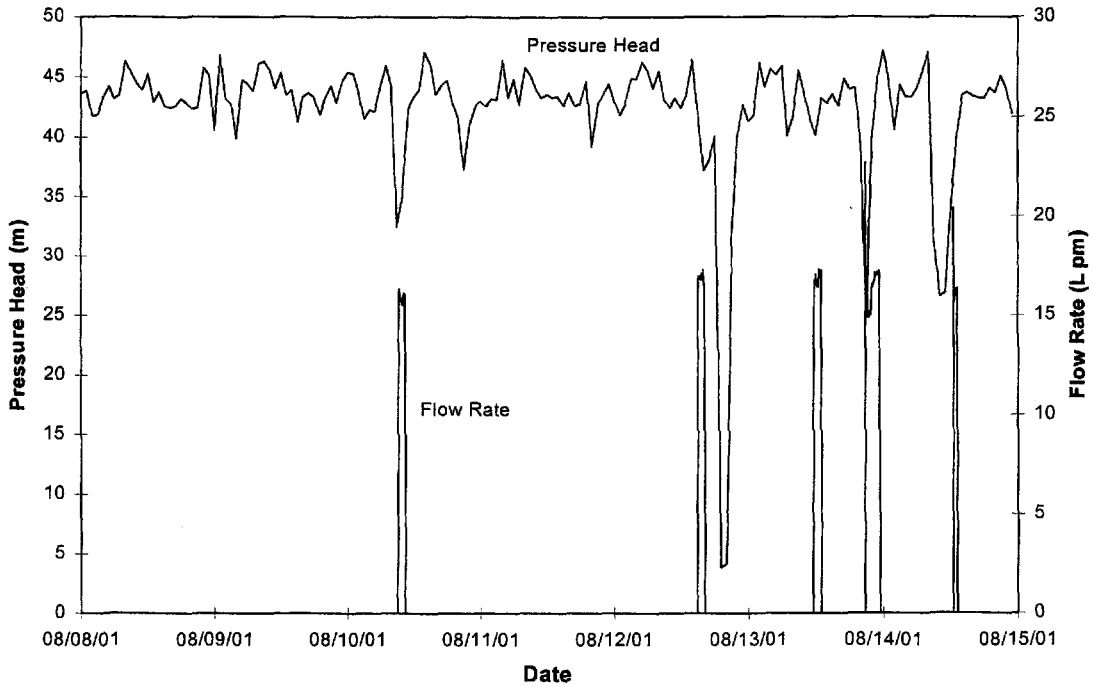

Figure 5.15 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Midpoint user from August 8 to 15, 2001.

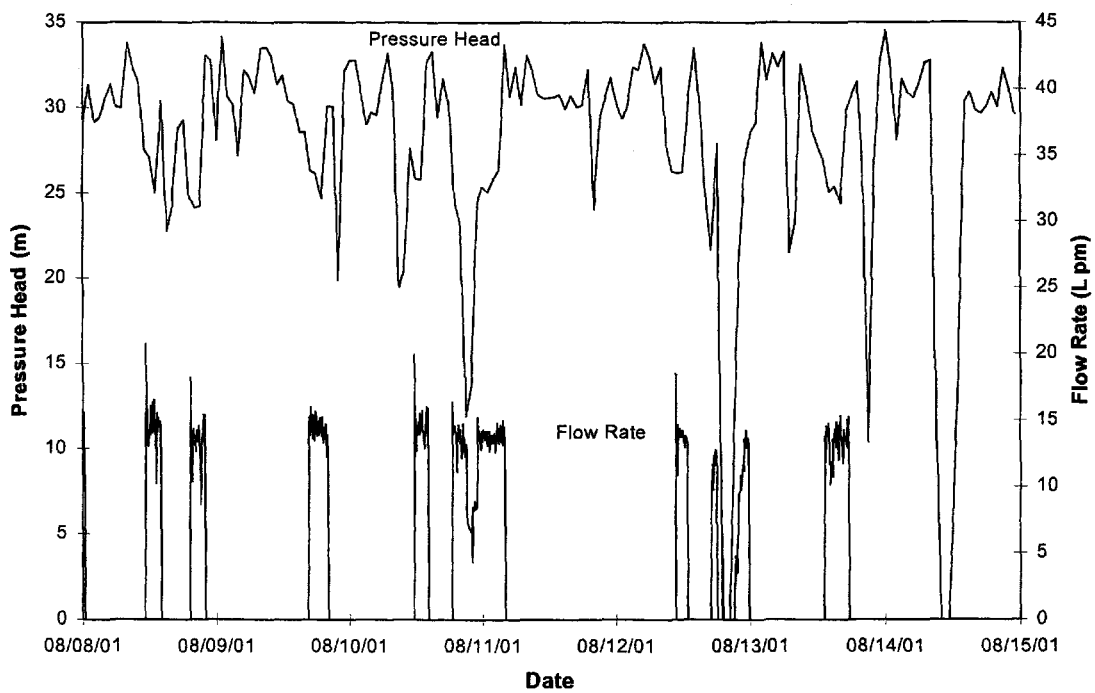

Figure 5.16 Example of flow rate and hourly pressure head recorded at the Taylorside/ Ethelton Farpoint user from August 8 to 15, 2001.

Table 5.5 Summary of flow and pressure head values from August 8 to 15, 2001 for each of the monitoring sites following an increase in system feed pressure to the Taylorside/Ethelton network.

Location	Max. Flow Rate (Lpm)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Booster Station	142[1]	48 4.67[3]	6	53
Midpoint Site	16[2]	42 5.45[3]	4	47
Farpoint Site	14[2]	28 6.75[3]	< 0	35

[1] Hourly flow rate.
[2] Cistern fill rate.
[3] Standard deviation.

5.2 Lucky Lake North Branch Pipeline

Flow and pressure at the upstream end of the Lucky Lake North Pipeline was calculated based on the data collected by the local Pipeline Association at the Booster Stations on the main regional pipeline. The connection to the Lucky Lake North Branch of the regional system is located between Booster Stations #3 and #4 (see Figure 3.2). The flow rate into the branch line was calculated as the difference between the two Booster Station flow rates, and pressure was calculated as the average of the Booster Station #3 outlet and the Booster Station #4 inlet pressures. A turbine flow meter and pressure transmitters at the Booster Stations provided instantaneous readings of the flow rate and pressure at 15 minute intervals. The monitoring equipment was not capable of reporting an average flow over the interval. Periodic failure of these devices resulted in significant gaps in the data. More notably, the timing of the readings taken by the flow sensors could not be synchronized. Calculations of hourly average flow rate into the branch network based upon the unsynchronized readings resulted in negative flow values. Therefore calculation of flow rates at a resolutions higher than a daily average are considered unreliable. Figure 5.17 shows the daily average flow rate and pressure head data collected during the study. Table 5.6 summarizes the daily data.

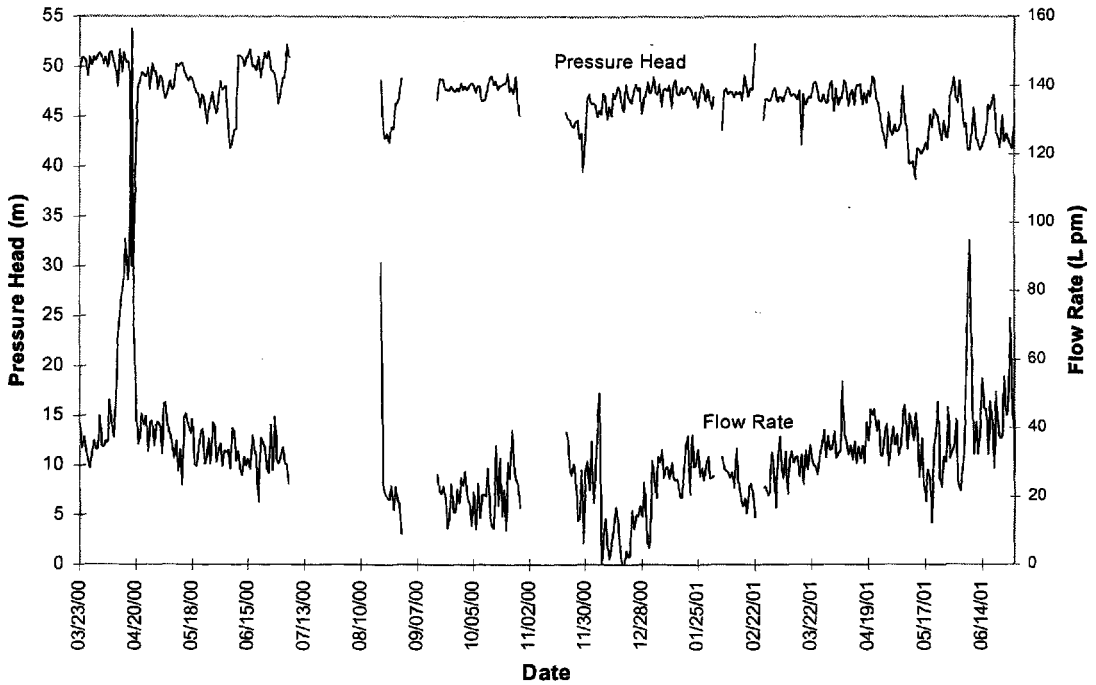

Figure 5.17 Calculated daily average flow rate and pressure head into the Lucky Lake North Pipeline from Mid-March 2000 to Mid-July 2001.

Table 5.6 Lucky Lake North characteristic daily average values for the period of study.

Parameter	2000 (March 23 to December 31)	2001 (January 01 to July 04)
Average Flow (Lpm)	27	33
Peak Flow (Lpm)	88	95
Average Pressure Head (m)	48	46

As with the Taylorside/Ethelton pipeline, two users were selected for monitoring. The data loggers at these locations were also set up to average the data at the end of a five minute interval. A flow rate control device was part of the service connection at each user site, thus average cistern fill rates are limited at both sites.

The Midpoint site is located at a significantly lower elevation than the Booster Stations and the Farpoint site which is reflected in the recorded pressures. Pressure head and flow rate for a typical week in 2000 at these sites is shown in Figures 5.18 and 5.19.

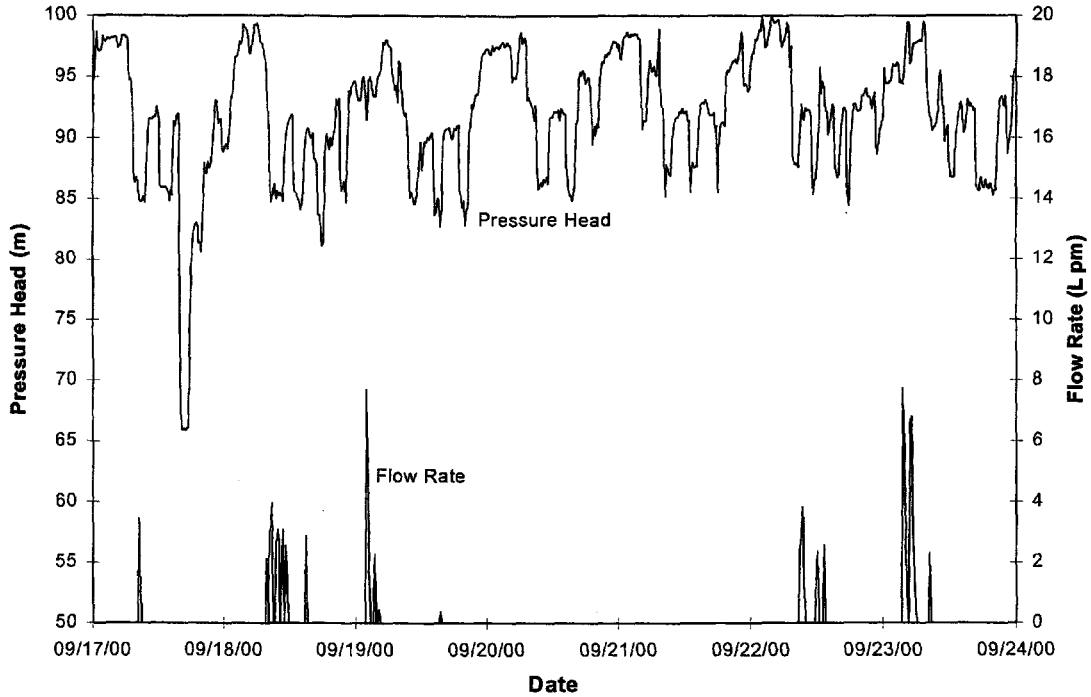

Figure 5.18 Example of flow rate and pressure head variation recorded at the Lucky Lake North Midpoint for a typical week in 2000.

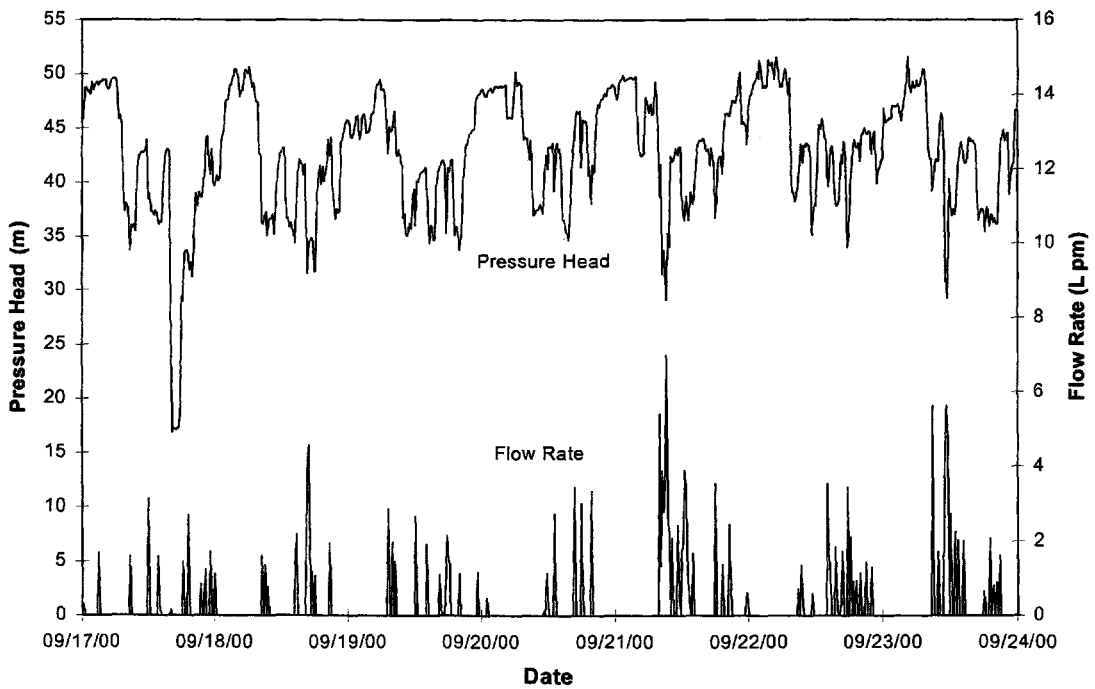

Figure 5.19 Example of flow rate and pressure head variation recorded at the Lucky Lake North Farpoint for a typical week in 2000.

Figures 5.20 and 5.21 illustrate the flow and pressure head variation recorded at the two user sites for a week with high demand in 2000. The increased demand elsewhere in the system and the low pressure at the Farpoint site during cistern filling increased the duration of one cistern fill to nearly 10 hours, as opposed to the one to three hour duration observed during the typical week.

The data for 2001 showed increased usage at the Farpoint site. Increased demand is evident elsewhere in the network as indicated by the pressure fluctuations recorded. Tables 5.7 and 5.8, quantify the variation and summarize the data shown in Figures 5.18 to 5.23. The Midpoint site service connection was noted to have a high concentration of particulate matter early on in the study. In 2001, the flow meter at the Midpoint site stopped accurately reporting flow. Despite the flow meter inaccuracy, the datalogger continued to produce electronic data files. The inaccuracy was not noticed until analysis of the data files was conducted at the end of the study. The water meter was removed and found to have particulate matter packed into the bearing surface of the meter paddle. As a result, flow data for the Midpoint site are unavailable for much of 2001.

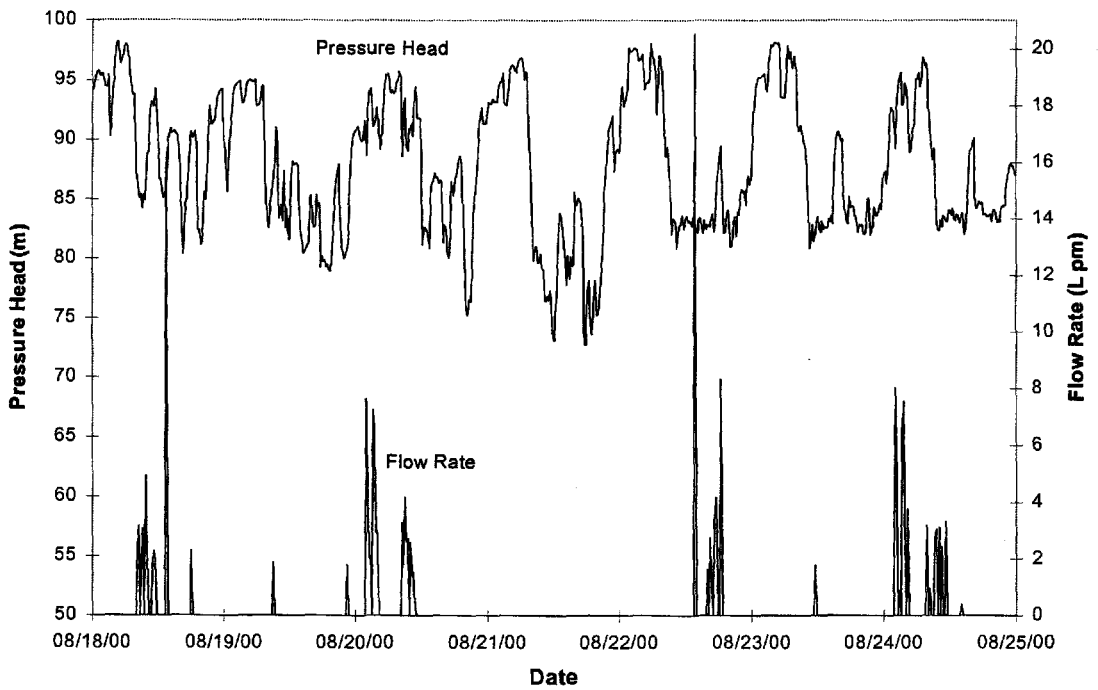

Figure 5.20 Example of flow rate and pressure head variation recorded at the Lucky Lake North Midpoint for a week with high demand in 2000.

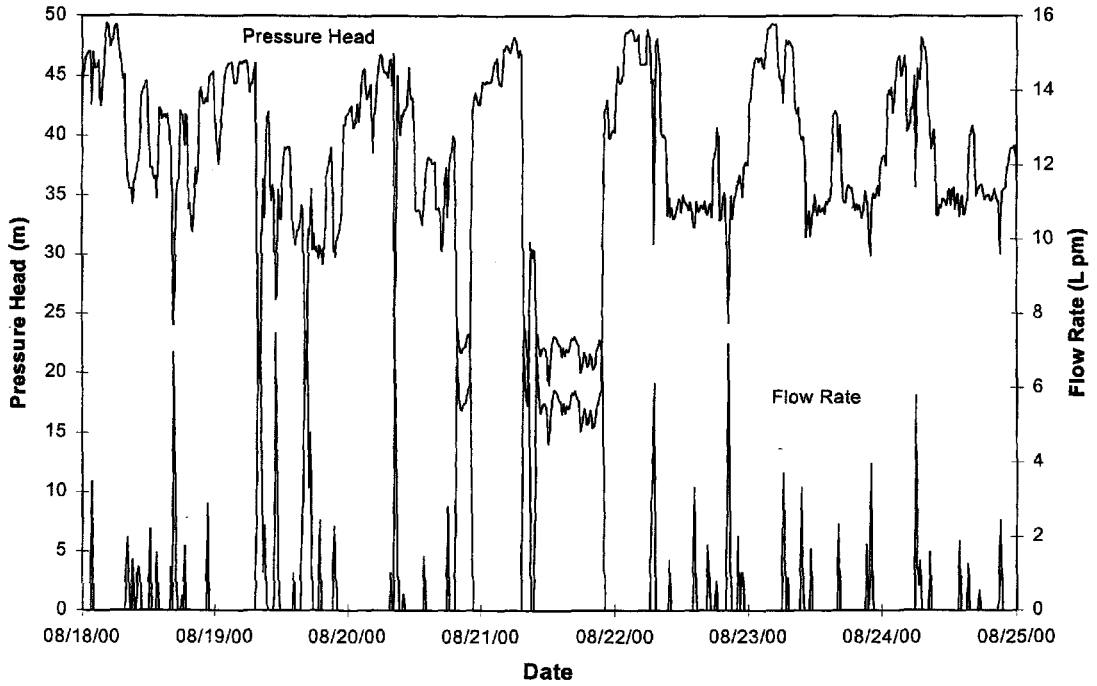

Figure 5.21 Example of flow rate and pressure head variation recorded at the Lucky Lake North Farpoint for a week with high demand in 2000.

Table 5.7 Summary of cistern fill rate, consumption and pressure head values for a typical week and a week with high demand in 2000 and 2001 for the Lucky Lake North Midpoint site.

Period	Fill Rate (Lpm)	Consumption (L)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Typical Week - 2000 (Sept. 17 to Sept, 24, 2000)	7.6	2,200	92 5.26[(1)]	66	100
Week with High Demand - 2000 (Aug. 18 to Aug. 25, 2000)	7.6	2,900	88 5.86[(1)]	73	98
Typical Week – 2001 (Mar. 8 to Mar. 15, 2001)	n.a.	n.a.	89 6.30[(1)]	67	99
Week with High Demand – 2001 (June 6 to June 13, 2001)	n.a.	n.a.	86 5.87[(1)]	73	97

n.a. – Data not available due to meter failure.
[(1)] Standard deviation.

Table 5.8 Summary of cistern fill rate, consumption and pressure head values for a typical week and a week with high demand in 2000 and 2001 for the Lucky Lake North Farpoint site.

Period	Fill Rate (Lpm)	Consumption (L)	Avg. Pressure Head (m)	Min. Pressure Head (m)	Max. Pressure Head (m)
Typical Week - 2000 (Sept. 17 to Sept, 24, 2000)	7.6	3,000	43 5.55[1]	17	52
Week with High Demand - 2000 (Aug. 18 to Aug. 25, 2000)	7.6	10,200	37 8.49[1]	14	49
Typical Week – 2001 (Mar. 8 to Mar. 15, 2001)	7.6	21,200	39 9.95[1]	14	51
Week with High Demand – 2001 (June 6 to June 13, 2001)	7.6	16,400	34 9.25[1]	14	48

[1] Standard deviation.

Figure 5.22 shows pressure head variations recorded at the Midpoint and Farpoint sites as well as the Farpoint flow for a typical week in 2001. Increased demand at the Farpoint site occurred as a result of livestock watering. A distinct drop in pressure during the filling of the Farpoint cisterns is evident and can be observed at the Midpoint site as well. This suggests that high demands at the edges of the network may have a strong influence on pressure at locations upstream in the system.

Figure 5.23 presents the patterns recorded for a week with high demand in 2001. The effect of high system demand is evident in the lower average pressure head at the two user sites (refer to Figures 5.22 and 5.23 as well as Tables 5.7 and 5.8), and the pressure head fluctuations at the end point of the system (Farpoint site). These fluctuations and lower pressure are apparent in the duration of cistern filling at the Farpoint on June 7, 2001 and the corresponding drop in pressure, which can be observed at the Midpoint site. The low number of users on the Lucky Lake North Branch line causes the effects of filling by an individual user to be more noticeable than in the Taylorside/Ethelton pipeline, which serves many more users; any combination of which could be drawing water.

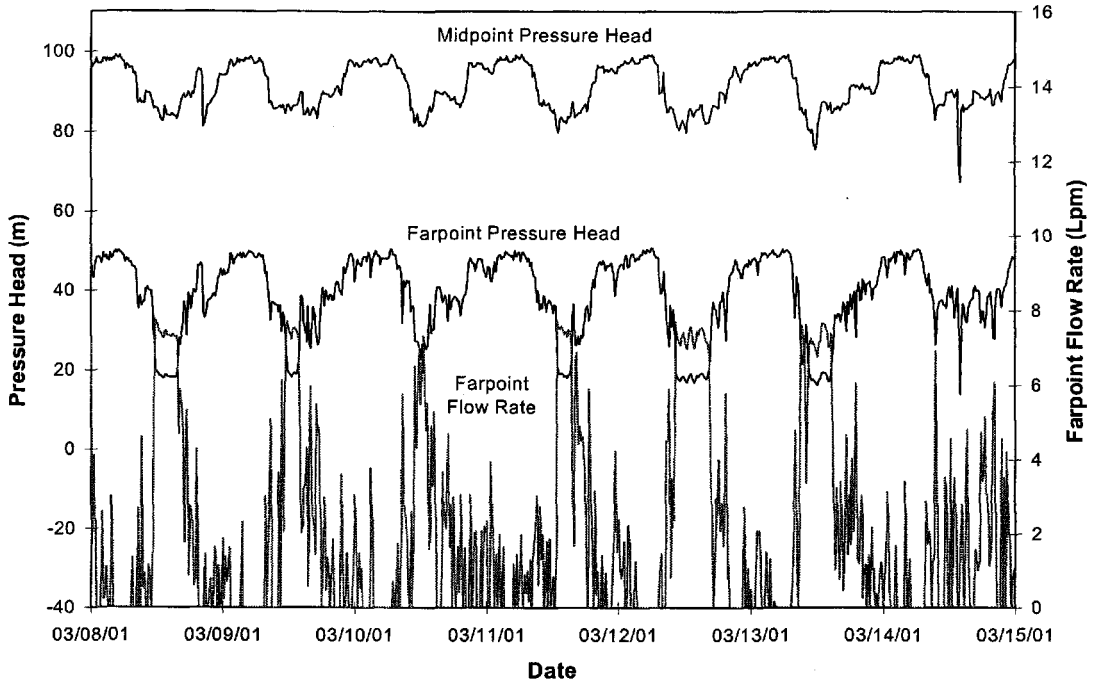

Figure 5.22 Example of flow rate and pressure head variation recorded at the Lucky Lake North Farpoint user and pressure head variation at the Midpoint for a typical week in 2001.

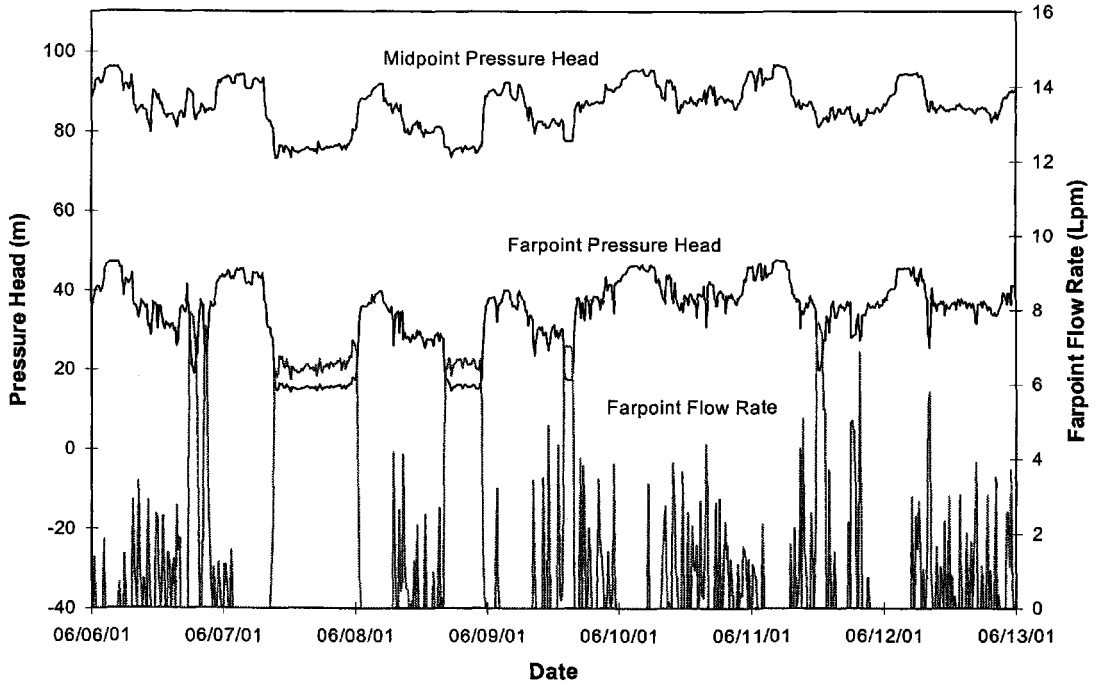

Figure 5.23 Example of flow rate and pressure head variation recorded at the Lucky Lake North Farpoint user and pressure head variation at the Midpoint for a week with high demand in 2001.

5.3 Flow and Pressure Characteristics

5.3.1 Seasonal System Demand Variations

The daily flows in the Taylorside/Ethelton pipeline had the highest peak in late summer (August to September) each year. The peak flows were recorded as a maximum hourly flow rate of 130 Lpm on September 30, 2000 and 142 Lpm on August 10, 2001. The average daily flow rates observed at the Booster Station were 25.0 and 29.3 Lpm in the 3rd quarters of 2000 and 2001, respectively. This increase is attributed to a lack of precipitation during the summer, which may have forced users to rely on the distribution system for lawn, garden and livestock watering rather than surface water sources (i.e. dugouts).

The demand on the system was also substantial during the winter due to cattle watering, and in early spring when the growing season had begun and cattle had not yet been put out to pasture. The average daily flow rate reached 38.4 Lpm in early spring. The lowest usage was in the latter part of the year in October and November, when the average flow rate was around 22.6 Lpm. Hourly flow rates showed a random pattern with several peaks occurring over the course of the day. This pattern is in contrast to urban systems that generally have a consistent diurnal pattern with two peaks each day.

Similar trends were observed on the Lucky Lake North branch pipeline. Using the available data, but removing peak flows caused by breaks and the subsequent flushing, the average daily flow rates were also found to decrease from early spring to the beginning of winter. This pattern was followed by an increase in demand due to wintering cattle in the late stages of winter and in early spring.

5.3.2 Peak Factors

The data collected from the Taylorside/Ethelton Booster Station was used to calculate the average and maximum flow rates and the peak factors commonly used in distribution system design. The results of this analysis are presented in Table 5.9. The

average maximum daily peak factor (maximum daily/average daily) for the Lucky Lake North Branch pipeline is 1.9. An accurate assessment of the hourly peak factor for the Lucky Lake North branch is not possible due to the missing data and flow monitoring timing issues.

Table 5.9 Rural peak daily and rural peak hourly factors observed at the Taylorside/ Ethelton pipeline.

Parameter	2000 (June 22-Dec. 31)	2001 (Jan. 01-Sept. 30)	Study Period (June 22, 2000-Sept. 30, 2001)
Maximum Hourly Flow (Lpm)	130	143	143
Maximum Daily Flow (Lpm)	53.6	62.2	62.2
Average Daily Flow (Lpm)	25.6	35.3	31.5
Max. Hourly/Avg. Daily	5.1	4.0	4.53
Max. Daily/Avg. Daily	2.1	1.8	1.97

In comparison, the daily peak factor for Saskatoon is about 2.1 and the hourly peak factor is around 3.1 (City of Saskatoon, 2003). Saskatchewan Watershed Authority (2004) reports a decrease in the peak factors with increasing population. This trend may be due to such factors as higher leakage in large systems and a larger number of industrial and commercial users that draw water at a more consistent rate.

Assuming an average of four persons per service connection on the Taylorside/ Ethelton network, the total population served by the system is estimated to be approximately 176. Saskatchewan Watershed Authority (2004) recommends a peak factor of 4.5 (hourly) and 3.0 (daily) for a population under 500 persons. The Taylorside/Ethelton system closely matches the recommended maximum hourly peak factor, but has a much lower maximum daily peak factor. Typically the users in the communities reported upon by Saskatchewan Watershed Authority (2004) have a direct

connection to a pressurized system, and do not use cisterns like the study sites. Therefore, the comparison of these peak factors illustrates that the low flow, low pressure philosophy and use of cisterns reduce the daily peak flow rates as intended, but do little to reduce hourly peak flows, likely due to the absence of flow control devices at the user sites.

5.3.3 Pressure Head

Common practice in design of distribution networks is to maintain a residual pressure greater than 140 kPa (14.1 m pressure head) at any point in the system during the design demand event (Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers, 1997) . The low pressures and large variations in pressure recorded at the Taylorside/Ethelton monitoring sites indicate that the system demand often exceeds the pump capacity. The low pressures suggest that the pump is operating on the far right side of a typical pump curve where a small increase in demand can result in a large decrease in pressure. Low pressures could also be related to the operating pressure of the regional water line. The Taylorside/Ethelton Booster Station pump discharge pressure is highly dependent on the regional pipeline pressure. The region received little rainfall in the summer of 2001, which may have resulted in high demand in the regional network, which in turn may have led to low inlet pressures and the low pressure observed in the Taylorside/Ethelton branch.

Variations in pressure were most pronounced at the peripherals of the two rural pipelines studied. This pressure drop was particularly notable when the users at the extents of these networks were drawing water. The long pipe lengths, which are typically small diameter at the system peripherals (<50 mm), contribute to large pressure drops through frictional losses. The Farpoint site in the Lucky Lake North branch pipeline recorded a drop in pressure head of 20 m below the average pressure head during cistern filling that coincided with periods of peak system demand.

Increased demand upstream of users at the system periphery can also reduce the residual pressure at these locations. The reduced pressure was evident by lower system pressures recorded at the Farpoint locations in each network during periods of high

system demand, when the Farpoint locations were not filling the cisterns. Increased demand upstream results in additional headloss, leaving a lower residual pressure for users downstream.

During the week with high hourly demand presented for the Taylorside/Ethelton pipeline (June 14 to 21, 2001), pressures at the Farpoint site dropped to below the acceptable limit on several occasions. During the same period, the Midpoint site shows similar decreases in pressure. Several of these low pressure events (refer to Figures 5.5, 5.12, and 5.13) were analyzed to determine the approximate location of the major demand. During these events neither the Midpoint nor the Farpoint were filling the cisterns and the difference in pressure between these two sites was due to change in elevation alone. The results of these analyses suggest that in order for the pressure to drop to the levels observed at the Farpoint location, the major demand must be coming from near the Booster Station.

After the system pressures were increased in the Taylorside/Ethelton pipeline in August of 2001, there still appeared to be problems with low pressure (refer to Figures 5.14, 5.15, and 5.16). On several occasions, the Farpoint site of the Taylorside/Ethelton line recorded gauge pressures less than zero. These extreme low pressure occurrences are believed to be the result of a pump shutdown in the regional pipeline and maintenance activities in the branch pipeline and resulted in the hydraulic grade line elevation dropping to below the ground elevation of the Farpoint. Regardless of the cause, these unacceptably low pressures, as discussed in Chapter 2, could result in contamination of the distribution system by drawing water into the pipeline through points of leakage. Further, when the pressures were increased in August of 2001, the flow rates into the Midpoint site increased, suggesting the increase in pressure only served to provide more flow to users nearer to the Booster Station as cistern fill rate is not regulated in the Taylorside/Ethelton network. While pressures at the Farpoint remained higher on average, there were still very low pressure occurrences, likely due to upstream use.

The use of flow restriction devices at sites near the Booster Station could improve the pressure residual in other parts of the system by reducing the frictional headloss in the upstream end of the network. The pressure at the users will still drop when flow is routed

to the users at the extents of the network due to headloss in the small pipes but the driving pressure at the upstream end of the service line will be higher. The flow restriction devices will increase cistern fill times. The increased fill times may even result in increased pumping cost, as the pump will be required to run longer at a fixed speed and at a lower efficiency to supply the same volume. Another option may be to increase the size of the pump, as this system in its current state, appears to be nearing the limits of its capacity.

Flow restriction is employed in the Lucky Lake North branch. On this pipeline, although the pressure does decrease throughout the system, particularly during filling at the Farpoint, the residual pressure remains at an acceptable level, regardless of demand elsewhere in the pipeline. Another difference between the two pipelines studied that may affect pressure fluctuation is the drawdown level in the cisterns before actuation. The users on the Lucky Lake North branch employ smaller drawdown levels, causing them to come online more frequently for a shorter duration.

Maintenance of residual pressures can be a complicated problem. Determining the cause and correction of chronic low system residuals such as those observed in a rural pipeline network would require extensive monitoring and a model simulation of an extended period. The model may then be used to study the methods employed to improve system pressure residuals. In the context of rural water distribution supply, the difficulties surrounding the maintenance of pressure residuals warrants further, more detailed study.

5.3.4 Quarterly Demand Patterns

Quarterly demand observed at the user sites on each of the networks studied presented an interesting comparison of agricultural operations. Table 5.10 lists the observed quarterly demand at the two sites on the Taylorside/Ethelton pipeline. The values shown are rounded to the nearest 1000 L.

Table 5.10 Observed quarterly demand at the Taylorside/Ethelton Midpoint and Farpoint monitoring sites.

Quarter	Midpoint Demand (L)	Farpoint Demand (L)
3-2000	71,000	116,000
4-2000	81,000	103,000
1-2001	82,000	365,000
2-2001	97,000	693,000
3-2001	76,000	258,000

The data listed in Table 5.10 illustrates the difference in water use between a farming operation (Midpoint) and medium sized livestock operation (Farpoint). Both locations have a similar number of household residents. The Farpoint had some livestock on site all year round but the largest increase in demand corresponded with the cattle being wintered at the site. Demand also increased significantly at the Farpoint in the late spring, when there is typically a population increase in livestock from calving and lawn and garden watering are occurring.

Table 5.11 shows the observed water usage on the Lucky Lake North Branch pipeline. Flow rate at these sites were generally limited to 7.8 Lpm by a flow regulator. In 2001, when the Midpoint pulse flow meter had failed, billing records have been substituted. When billing records were unavailable, an entry of 'N/A' is shown. As is the case with the Taylorside/Ethelton billing records, the values shown may not be an accurate reflection of the actual consumption for the period. The values shown are to the nearest 1000 L.

As for the Taylorside/Ethelton line, the Midpoint and Farpoint users on the Lucky Lake North Branch were farming and cattle operations, respectively. Table 5.11 shows a notable increase in demand during the winter months due to wintering of cattle and peak consumption in the 2nd quarter of 2001. It was observed that some of the livestock were

kept in the yard in 2001, and demand increased significantly in the 3rd quarter of 2001 over that observed in the 3rd quarter of 2000.

Table 5.11 Observed quarterly consumption at the Lucky Lake North branch Midpoint and Farpoint monitoring sites.

Quarter	Midpoint Demand (L)	Farpoint Demand (L)
3-2000	33,000	53,000
4-2000	43,000	153,000
1-2001	33,000	258,000
2-2001	N/A	284,000
3-2001	N/A	145,000

6.0 WATER QUALITY DATA, ANALYSIS AND DISCUSSION

The following sections present the water quality data collected over the study period of June 2000 to September 2001. Sampling was conducted at intervals of one to three weeks, depending on the level of activity observed (change in bacterial numbers and DOC) and anticipated (as a result of high temperatures) in the pipelines.

6.1 Taylorside/Ethelton Pipeline

The following subsections present plots of water quality parameters from samples collected from the Taylorside/Ethelton pipeline. On each trip to the site, samples were collected from the water entering the regional network from the water treatment plant in Melfort (sample from WTP discharge header), the Taylorside/Ethelton booster station, the Midpoint site and the Farpoint site.

6.1.1 Temperature

Figure 6.1 shows the temperature of the water in the pipeline at each site over the period of study. Yearly peak water temperatures occurred in August and September. The lowest temperatures were recorded in late March and early April. The most seasonal variability in temperature was noted at the water treatment plant. After water entered the pipeline, a cooling trend to the Taylorside/Ethelton booster station was noted. Typically there was little change in temperature throughout the remaining length of the pipeline. This cooling and temperature stability is attributed to the influence of the subsurface ground temperature and is enhanced by the long residence times in the system. Despite higher source water temperatures in the 3rd quarter of 2001, the water temperature in the pipeline remained relatively consistent with the 3rd quarter of 2000.

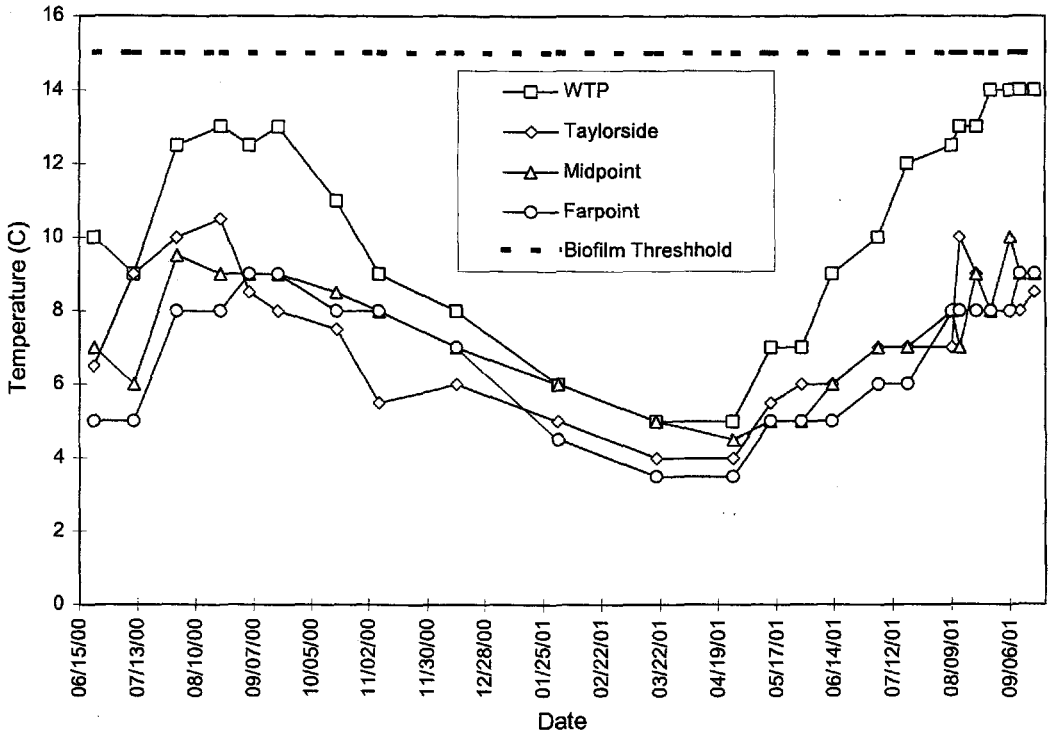

Figure 6.1 Water temperature measured at the Taylorside/Ethelton sites from June 2000 to October 2001.

Even at the highest recorded temperatures, the biofilm threshold value of 15 °C proposed by Piriou et al. (1998) was not exceeded. After transport through the pipeline from the Melfort water treatment plant to the Booster Station, water temperatures typically remained below 10 °C.

6.1.2 Dissolved Organic Carbon

Figure 6.2 shows the DOC concentration measured at each monitoring site. DOC varied seasonally with peak concentrations coinciding with high temperatures in August and September. Raw water was not sampled, but based on available literature, it is reasonable to believe that peak organic matter content and biological activity in the source water during these periods, likely result in increased DOC getting through the water treatment process and into the network. Figure 6.2 does not clearly indicate consumption or generation of DOC as the water travels through the network. Trends along the pipeline are difficult to discern due to the sample timing, which would not have

coincided with the movement of a discrete volume of water through the system. The lowest DOC concentrations and changes in DOC concentration along the pipeline coincided with the lowest water temperatures.

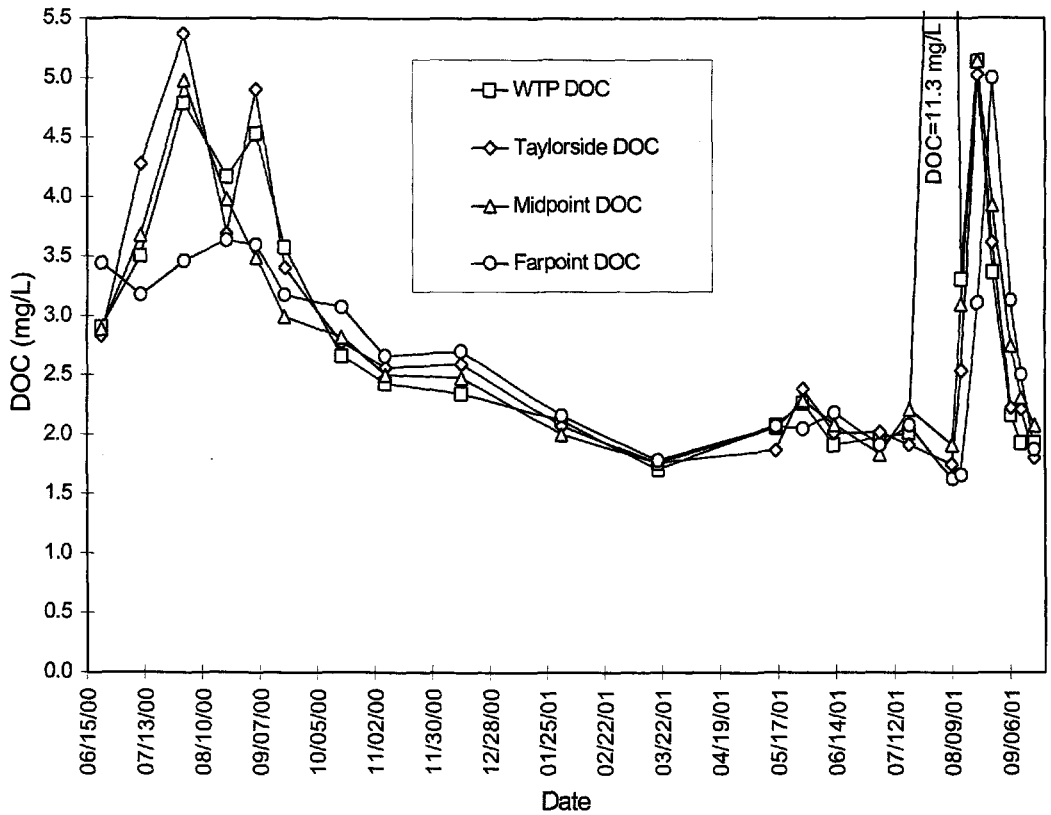

Figure 6.2 Dissolved organic carbon concentration measured at the Taylorside/Ethelton sites.

In July 2001, the Melfort water treatment plant had to switch to an alternate source of raw water for approximately 9 days when the primary supply main developed a break. This alternate source was a shallow, surface reservoir that collects local runoff. Local reservoirs of this type typically contain high concentrations of organic matter. As a result, the DOC concentration leaving the water plant rose dramatically (11.3 mg/L). Following the switch back to the primary source (Codette reservoir on the Saskatchewan River), a flood wave from a storm in Alberta arrived in the reservoir, which resulted in additional high DOC concentration water being fed into the network. The 'spike' of

DOC from these events was observed to move through the network in the following weeks.

6.1.3 Biodegradable Dissolved Organic Carbon

BDOC results are shown in Figure 6.3. The results are highly variable early in the study due to contamination of the samples by the distilled water used. A switch to ultra-pure water reduced the variability in subsequent testing. The only conclusion that can be drawn from comparing Figure 6.2 to Figure 6.3 is that the BDOC represents a relatively small portion of the DOC. The method used for determination of BDOC has been reported to not work well for concentrations less than 0.2 mg/l (Servais et al., 1989). Unfortunately, the substrate concentrations measured after the switch to ultra-pure water are very near this level. Consequently, negative values for BDOC (as shown in Figure 6.3) often resulted due to the poor sensitivity of the method at low BDOC concentrations. A slight increase in BDOC was evident in August and September of 2001. It is assumed that this increase was a result of the change in source water and increase in DOC.

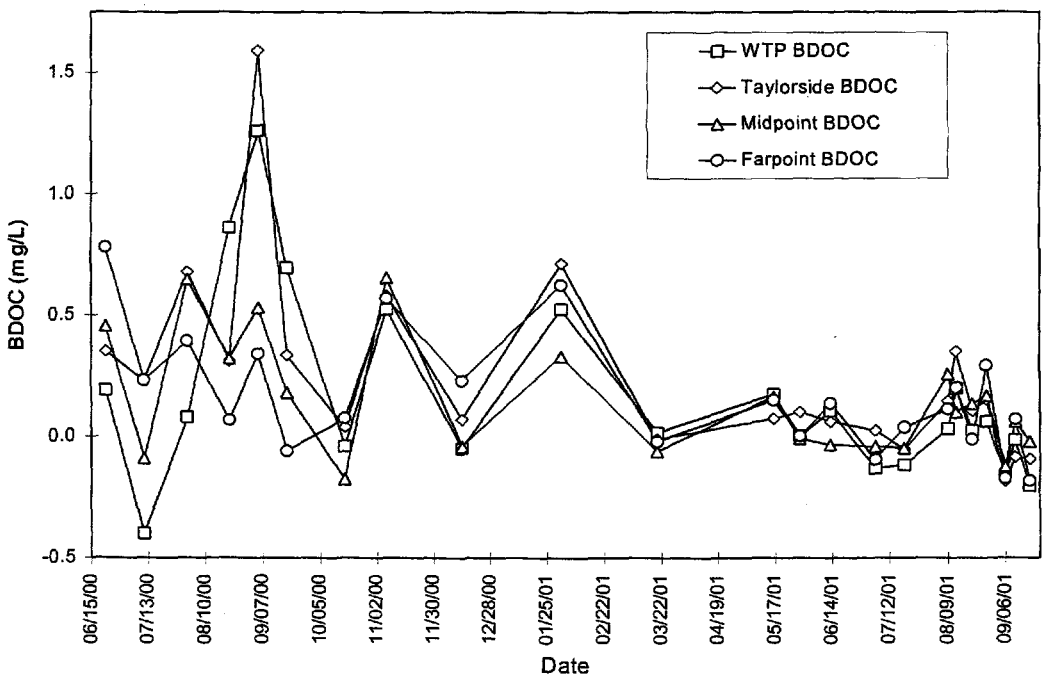

Figure 6.3 BDOC measured at the Taylorside/Ethelton monitoring sites.

6.1.4 Turbidity

Measured Turbidity is shown in Figure 6.4. Turbidity values typically remained below the limit of 1 NTU set out in the Canadian Guidelines for Drinking Water Quality (Health Canada, 2004). Seasonal variation showed an increase in magnitude and variability in late summer of each year. This increase coincides with periods of elevated biological activity, DOC, and temperature and may be the result of suspended microbes in the water column. The lowest readings occurred during the winter months. The change in source water in August of 2001 resulted in some of the highest readings recorded.

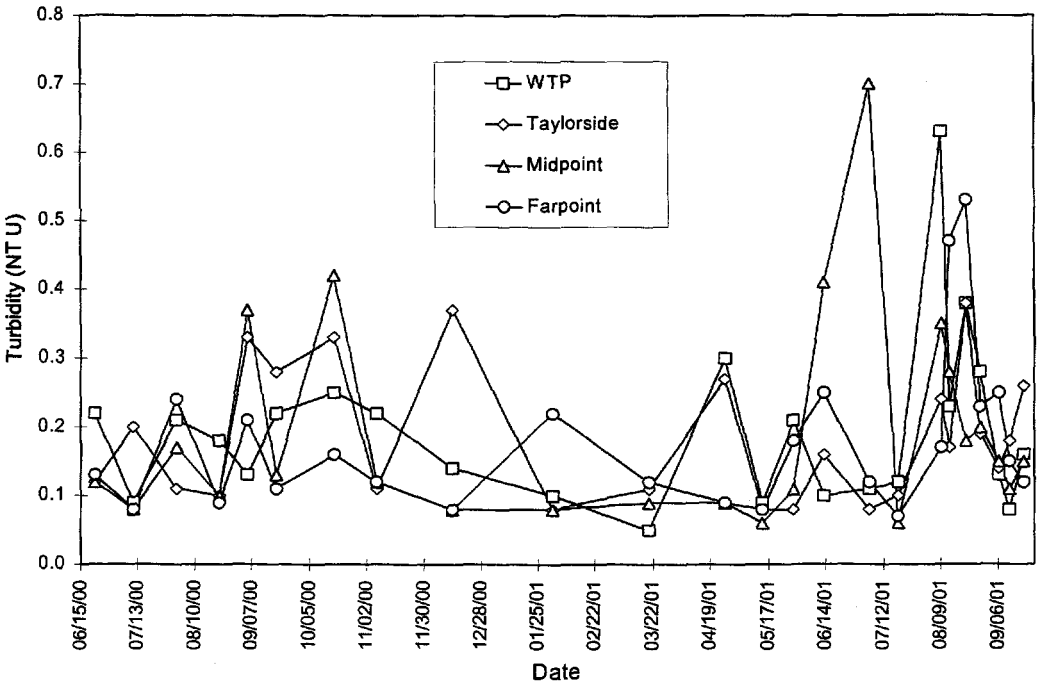

Figure 6.4 Turbidity measured at the Taylorside/Ethelton monitoring sites.

6.1.5 Epifluorescent Bacteria Counts

The bacteria counts presented in Figure 6.5 include both inactivated and viable bacteria within the bulk water in the pipeline. Peak bacterial numbers generally occurred when temperature and DOC were the highest, in August and September. The counts were the lowest and least variable during the periods with cooler water temperatures. An

exception to this trend occurred in April of 2001 when a sudden increase in numbers was noted. This event could have been a result of spring overturn in the raw water reservoir or possibly a slough of a biofilm elsewhere in the network. An increase in bacterial counts also occurred in August of 2001, coinciding with the passage of the DOC 'spike'. No clear trend of increasing bacterial numbers along the length of the line could be detected.

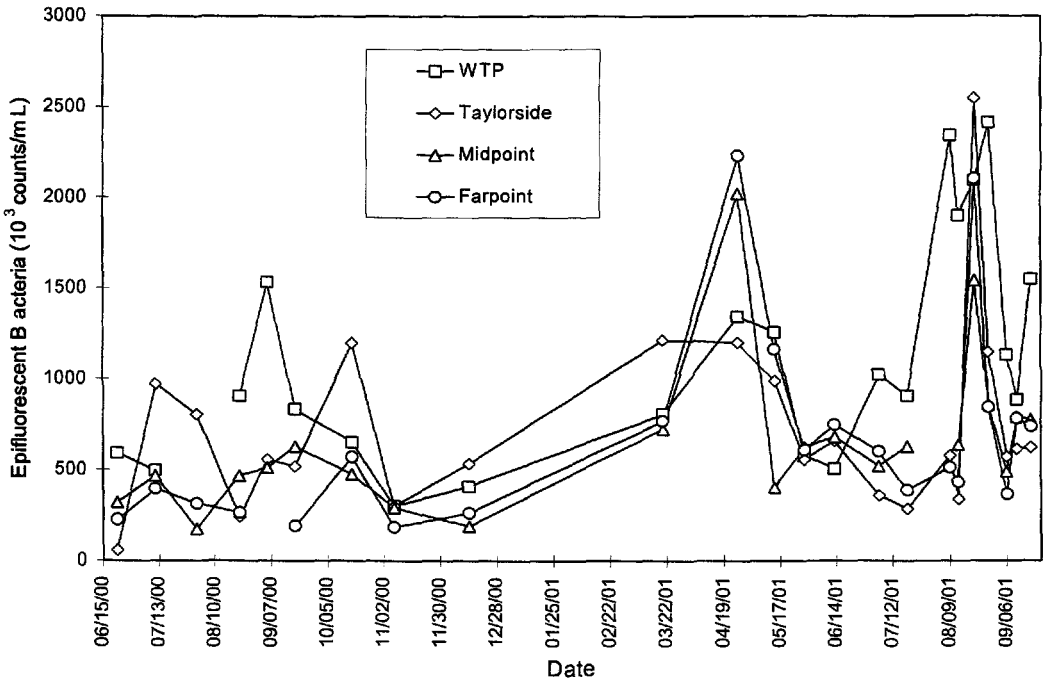

Figure 6.5 Epifluorescent bacteria counts observed for the Taylorside/Ethelton monitoring sites.

6.1.6 Chlorine Residual

Total chlorine (free chlorine and chloramines) and free chlorine (HOCl and OCl^-) concentrations are shown in Figures 6.6 and 6.7. The average daily chlorine concentrations at the water treatment plant were calculated from the Melfort water treatment plant records provided by the Saskatchewan Water Corporation. The chlorine residuals were measured at each site during sampling. The free chlorine concentrations were typically about 80% of the total chlorine residual. The free chlorine residual

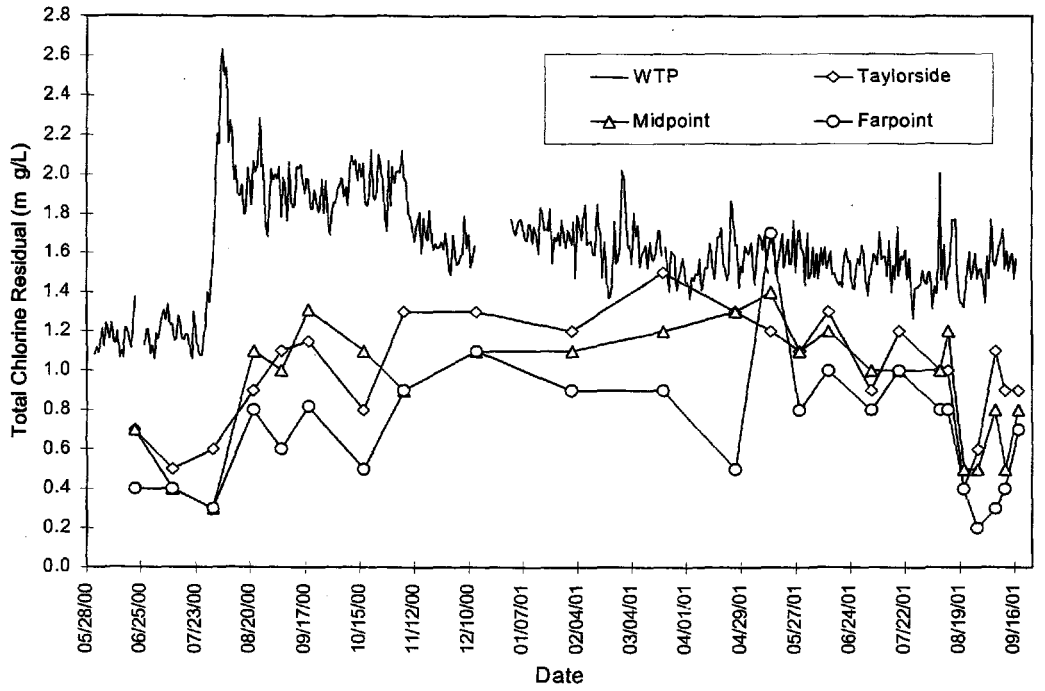

Figure 6.6 Total chlorine residuals measured at the Taylorside/Ethelton monitoring sites.

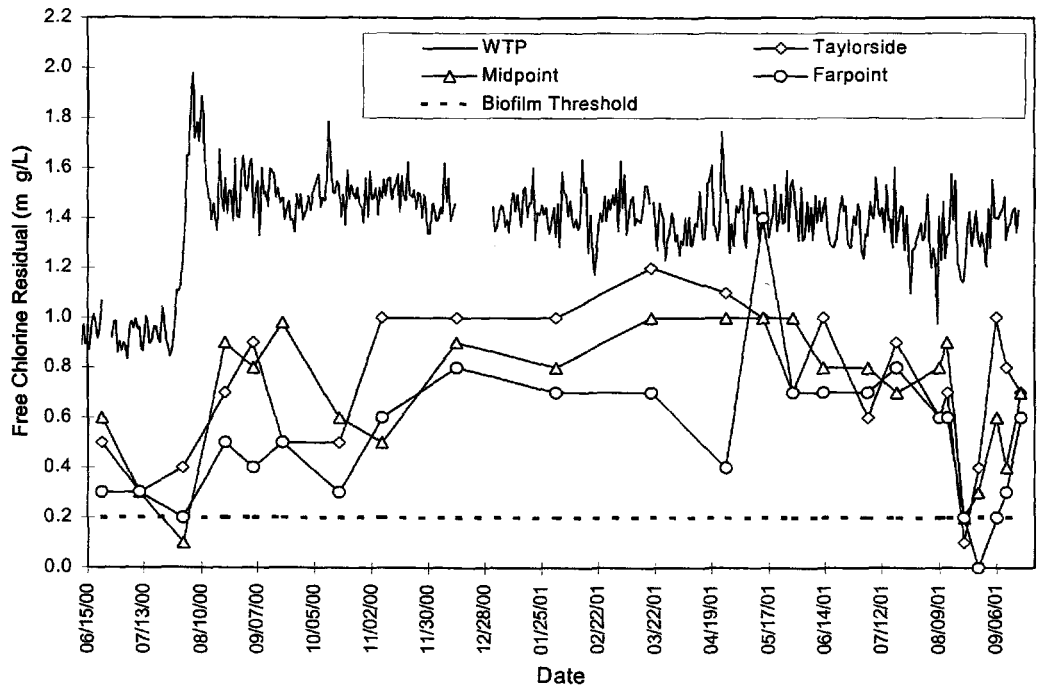

Figure 6.7 Free chlorine residuals measured at the Taylorside/Ethelton monitoring sites.

concentration fell below the biofilm threshold of 0.2 mg/l recommended by Piriou et al. (1998) twice during the summer months. In 2000 an increase in the chlorine concentration at the water treatment plant in Melfort corrected the low levels observed in the system. In 2001, the increased DOC concentrations related to the change in raw water source caused the residuals to fall to unacceptably low levels. Passage of the DOC spike from the system was followed by the recovery of chlorine residuals throughout the network.

As expected, chlorine concentrations decreased as water was transported through the system. This depletion was less pronounced during low temperatures when the system residuals were observed to be highest.

6.1.7 Heterotrophic Plate Counts

Figure 6.8 shows the results of HPC testing and free chlorine concentrations in August and September of 2001 at the Taylorside/Ethelton monitoring sites. Having observed the free chlorine residual drop below the recommended biofilm threshold in August of 2000, it was decided that HPC samples should be collected for the same period in 2001. The intent was to determine the effect of the high DOC, high temperature, low chlorine residual characteristics of this seasonal occurrence on the viable fraction of the bacteria counts. A total of six samples were collected from each site during August and September of 2001, and sent to the Saskatchewan Health Provincial Laboratory for analysis.

The viable fraction of bacteria appeared to be negligible in the presence of an adequate free chlorine residual. Viable organisms were noted only after residuals dropped below the recommended threshold value of 0.2 mg/l (Piriou et al., 1998). The concentrations observed were well below 500 CFU/mL (believed to be the level above which health of the consumers can be adversely affected). This demonstrates the role of the chlorine residual in controlling bacterial levels in the presence of available substrate. Once chlorine residuals were increased and adequate contact time had elapsed, the number of viable organisms decreased to below the detection limit.

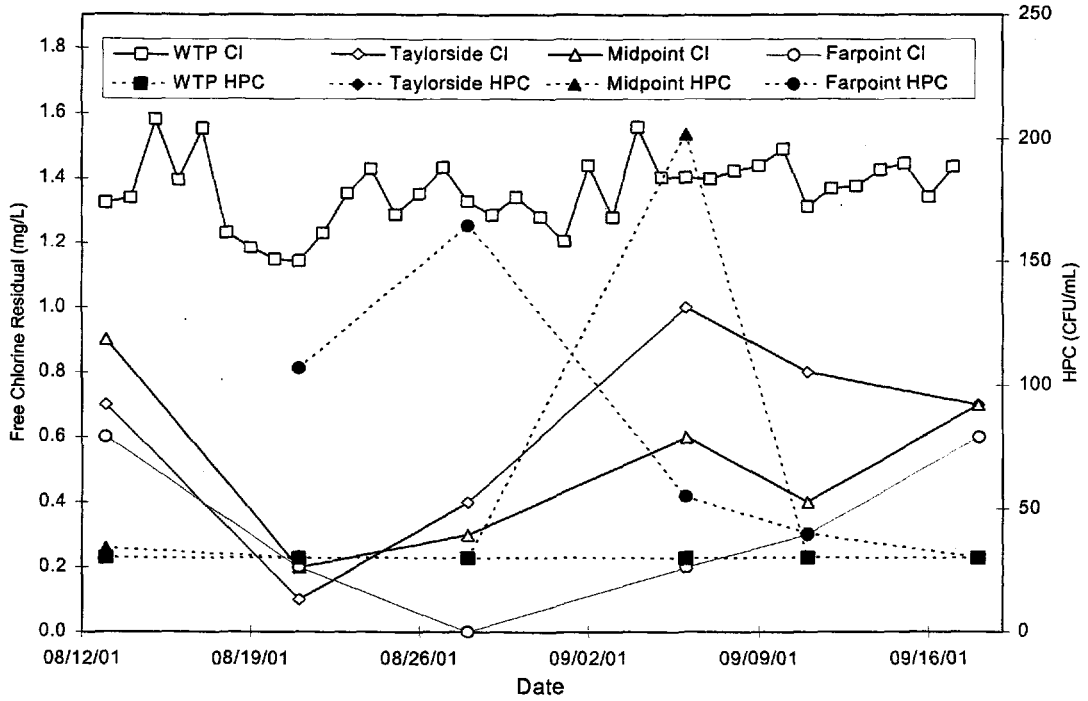

Figure 6.8 Heterotrophic plate counts and free chlorine residuals recorded at the Taylorside/Ethelton monitoring sites from August 13 to September 18, 2001.

6.1.8 Particle Size Analyses

Particle size measurements of suspended solids in the water samples are shown in Figures 6.9, 6.10, 6.11, and 6.12. Figure 6.9 shows the 2-5 μm range for the Taylorside/Ethelton monitoring sites and Figure 6.10 shows the 5-10 μm range. Particle size analyses (all ranges) for the Midpoint and Farpoint sites are shown in Figures 6.11 and 6.12, respectively. Particle size ranges of 2-5, 5-10 and 10-15 μm are of particular interest because they correspond to the size ranges of protozoan cysts. *Giardia* cysts are typically 6-14 μm in diameter and *Cryptosporidium* cysts are known to be 3-6 μm in diameter (Health Canada, 1999). Another cause for concern with particles entering the system is that bacteria attached to particles are believed to be more resistant to disinfection (LeChevallier, 1988a).

In 2000 the particle counter was returning unrealistic results and was sent away for calibration. It was returned to the lab early in 2001. The analyses completed after the analyzer was re-calibrated showed the majority of particles to be in the 2-5, 5-10 and 10-

15 μm size ranges. The switch to the alternate raw water source resulted in a large increase in the number of particles in the 2-5 and 5-10 μm ranges. The increased concentrations were noted at the Midpoint and Farpoint sites and are clearly evident in Figures 6.11 and 6.12. The relative magnitude of the concentrations between the two user sites suggests that these particles are not prone to settling in the pipeline. This suggests that the particles have a nearly neutral buoyancy, which is a characteristic of microbes. Turbulence within the pipeline bulk water as it flows may also help to keep small particles suspended, although the stagnant periods in the service lines between cistern fills should allow time for settling.

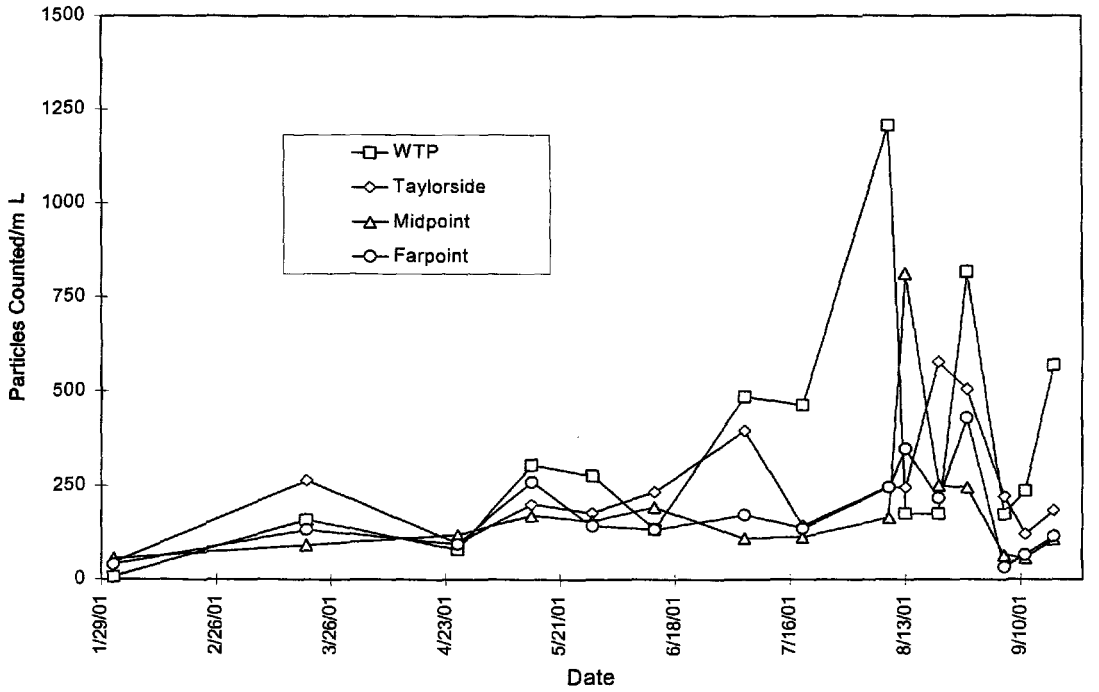

Figure 6.9 Particle counts for 2-5 μm particles as recorded at the Taylorside/Ethelton pipeline monitoring sites.

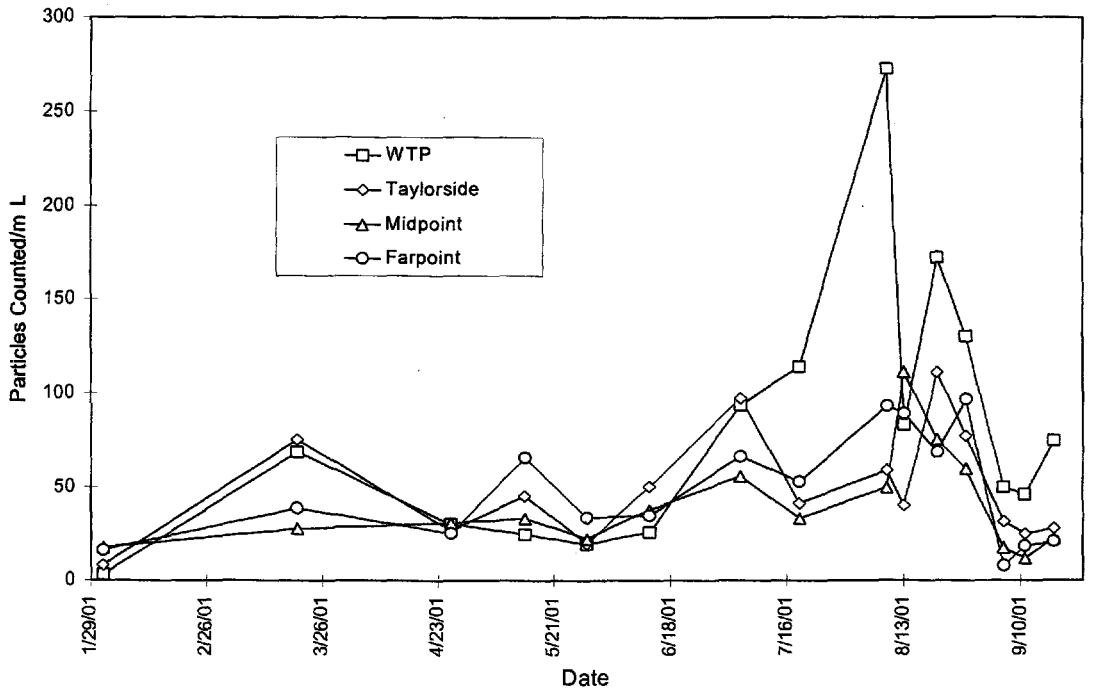

Figure 6.10 Particle counts for 5-10 μm particles as recorded at the Taylorside/Ethelton pipeline monitoring sites.

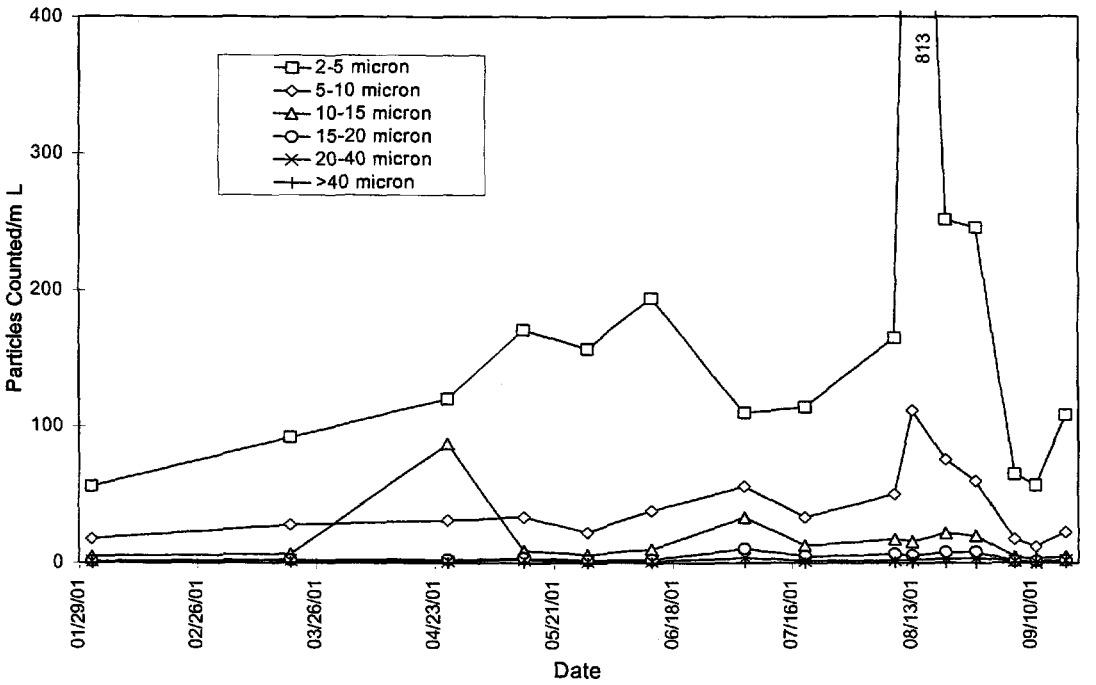

Figure 6.11 Particle size analysis completed for the Midpoint monitoring site of the Taylorside/Ethelton Pipeline for the period of February 1 to September 18, 2001.

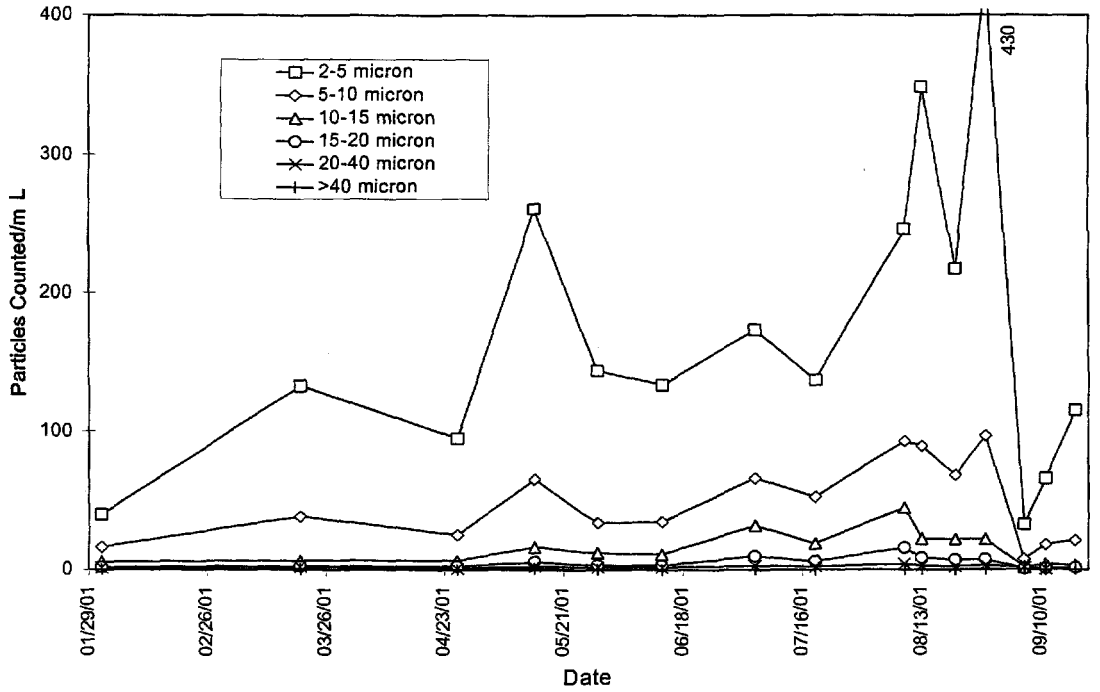

Figure 6.12 Particle size analysis completed for the Farpoint monitoring site of the Taylorside/Ethelton Pipeline for the period of February 1 to September 18, 2001.

6.2 Lucky Lake North Branch Pipeline

The following subsections present plots of water quality parameters from samples collected from the Lucky Lake North pipeline. Each trip involved sample collection from the intake pump wet well at the Saskatchewan Water Corporation irrigation pumping station on Lake Diefenbaker, or the nearest available point (typically booster station # 2 when the irrigation pumping station was closed for the winter), booster station #3, the Midpoint site and the Farpoint site.

6.2.1 Temperature

Raw water is drawn directly into the pump wet well from a shallow bay on Lake Diefenbaker. In contrast, the water reaching the Melfort water treatment plant, is subjected to the ground temperature surrounding the raw water pipeline prior to treatment.

Figure 6.13 shows the temperature data collected for the Lucky Lake North Branch pipeline. High water temperatures are evident at the source and at booster station #3. The residence time from the intake to booster station #3 is very short due to the high flow rates in the main required to supply the many users downstream of the Lucky Lake North supply point. Once into the Lucky Lake North branch pipeline, water flow rates decrease to those more characteristic of a small rural distribution system.

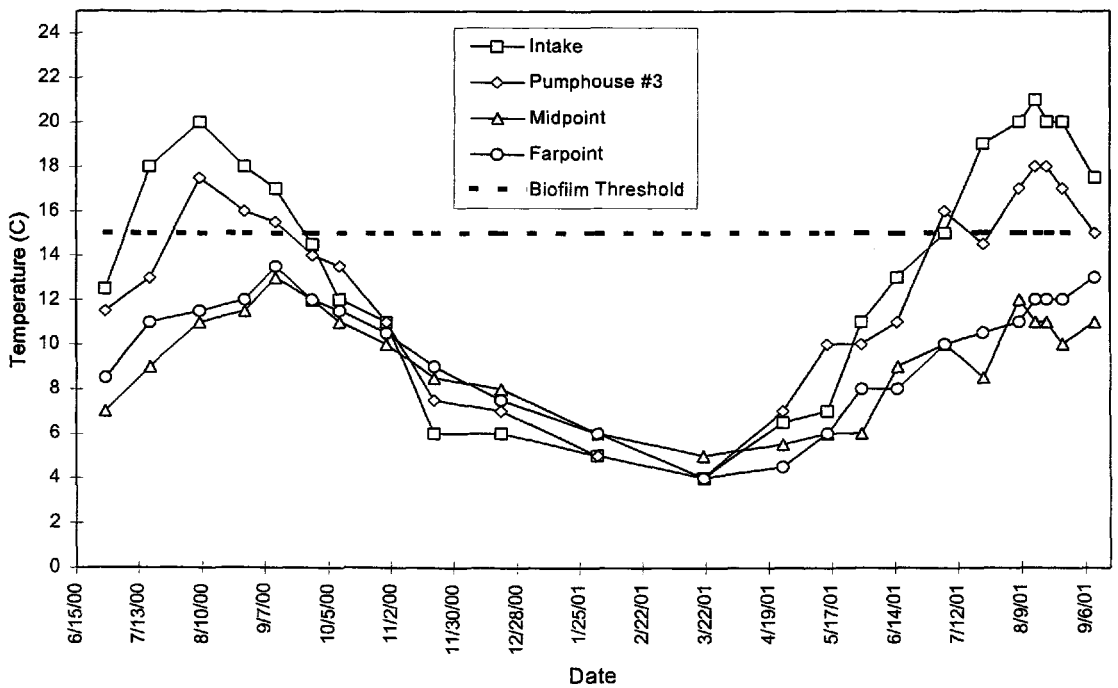

Figure 6.13 Temperature measurements taken at the Lucky Lake North monitoring sites.

The water temperature at the source peaked at 20 °C in August of 2000 and reached 21 °C in August 2001. Temperatures at booster station #3 also exceeded the biofilm threshold in parts of August and September with peak temperatures of 18 °C. As with the Taylorside/Ethelton pipeline, ground temperature cools the water in the pipeline. However, the elevated source water temperature resulted in the Midpoint and Farpoint site water temperatures peaking at around 13 °C. These high temperatures increase the potential for biofilm formation. Minimum temperatures around 4 °C were recorded in late March of 2001. Temperature was generally found to decrease with distance from the

intake, with the exception of November 2000 when the source water temperature dropped below the ground temperature and a warming trend was observed in the line.

6.2.2 Dissolved Organic Carbon

DOC measurements taken over the study period are shown in Figure 6.14. The peak source water concentrations occur in August and September of 2000. In 2001, the same peak concentration trend was not evident. In general the DOC concentrations tend to decrease with residence time. The magnitude of DOC concentration observed, and the change in DOC between sites (presumably due to consumption), was less during the colder months. The gap in the data is the result of exclusion of samples that were known to be contaminated.

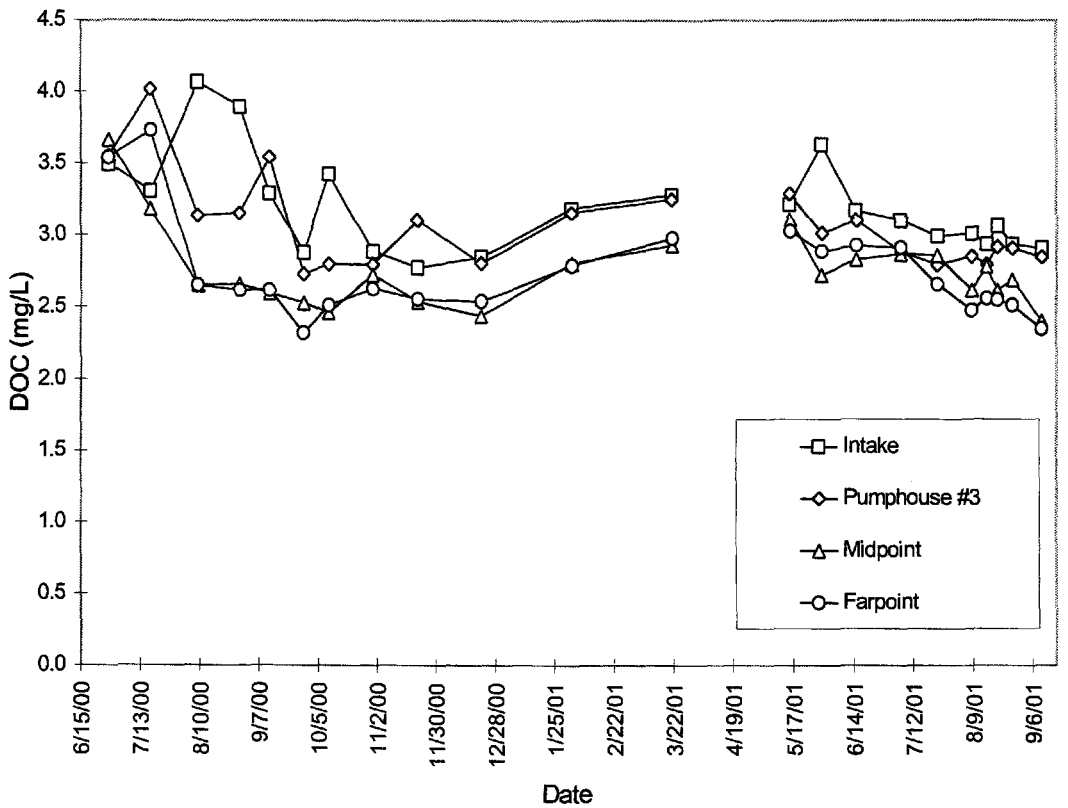

Figure 6.14 Dissolved organic carbon measurements collected from the Lucky Lake North monitoring sites.

6.2.3 Biodegradable Dissolved Organic Carbon

Figure 6.15 shows the results of the BDOC analyses for the monitoring sites. As with the Taylorside/Ethelton results the BDOC determinations for Lucky Lake are variable, and very low. This may be indicative of low concentrations or complications with the method used for determining the biodegradable fraction. As with the Taylorside/Ethelton line, a slight increase in the BDOC was observed during the warmer periods of the year suggesting a change in the fraction of bio-available substrate for different periods of the year.

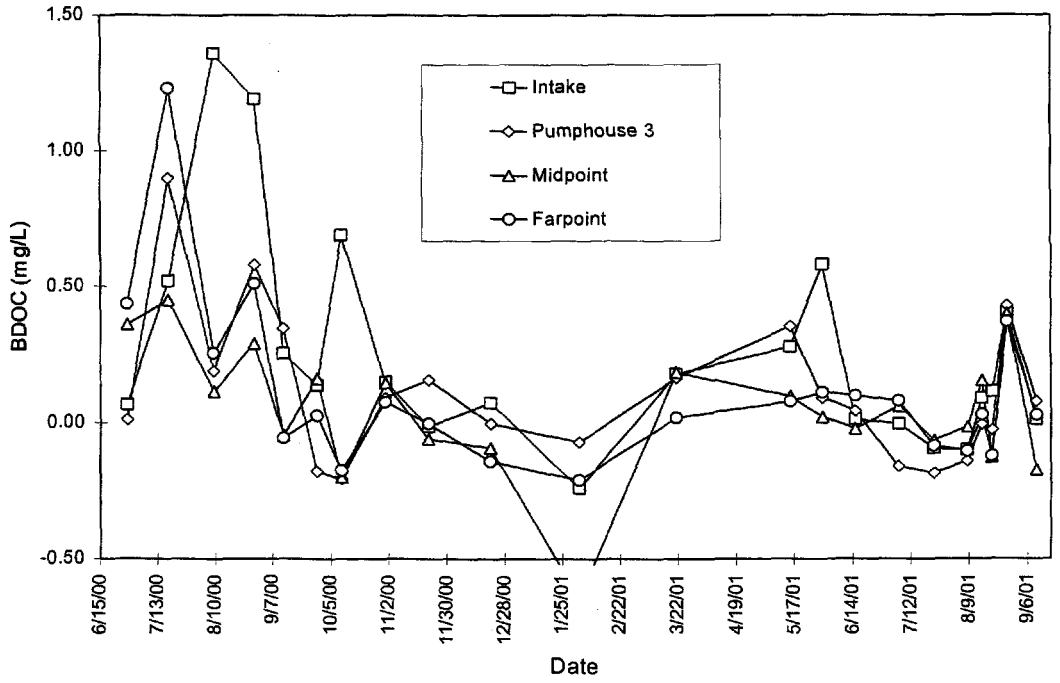

Figure 6.15 Biodegradable dissolved organic carbon measurements collected from the Lucky Lake North monitoring sites.

6.2.4 Turbidity

Figure 6.16 shows the turbidity values collected for each site. Turbidity appears to vary seasonally throughout the pipeline. Peak measurements occur during the high temperature periods. The measurements were typically above the recommended value of 1 NTU (Health Canada, 2004) as the water is untreated. The highest values were noted at the Midpoint monitoring site. Sediment deposited in the service line at the midpoint site

was re-suspended during sampling. Re-suspension was indicated by increased turbidity with higher flow rates during sampling. The Farpoint site consistently had low readings, likely due to the settlement of particulate matter upstream of the site.

During periods of high demand at the Farpoint site, the turbidity was observed to increase slightly, but remained low compared to measurements elsewhere in the system. As with the Taylorside/Ethelton pipeline, the minimum turbidity values were observed during the winter months. The levels at booster station #3 were observed to increase significantly during the peak temperature season. Wave action in the reservoir due to wind or recreational activities may also have had an effect on the turbidity of the raw water entering the network.

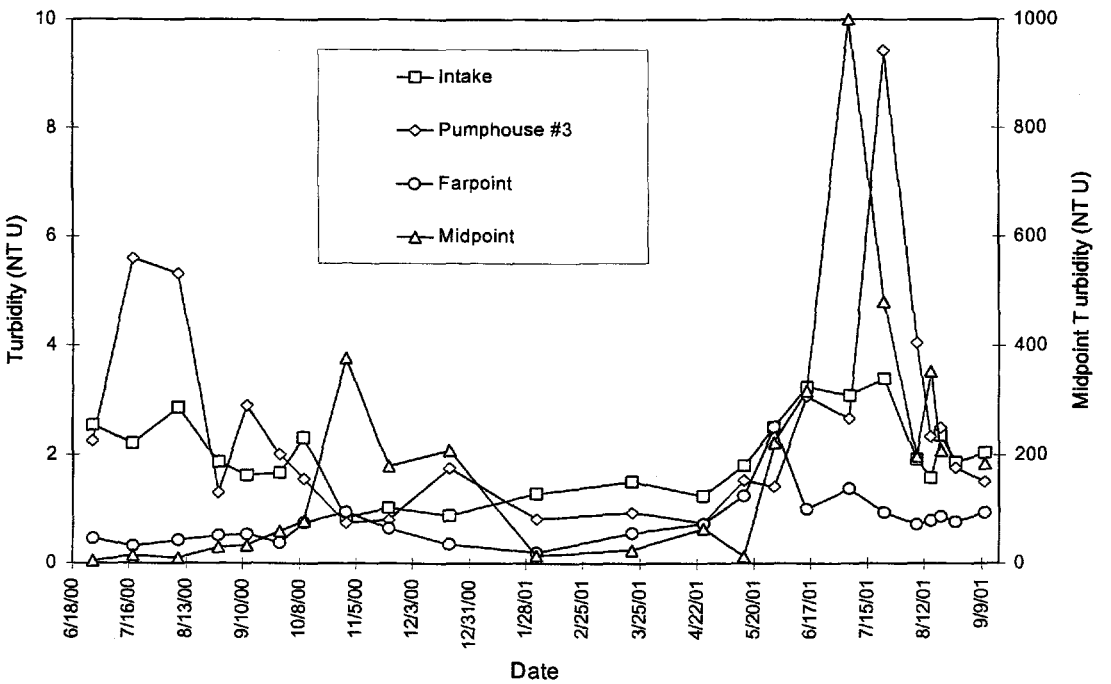

Figure 6.16 Turbidity measurements recorded at the Lucky Lake North monitoring sites.

6.2.5 Epifluorescent Bacteria Counts

Figure 6.17 shows the epifluorescent bacteria counts in the Lucky Lake North pipeline over the period of study. There is no consistent trend of bacterial growth with residence time in the pipeline. Peak activity was recorded in the summer of 2001, corresponding with the highest temperatures. A peak similar to that observed within the

Taylorside/Ethelton pipeline in late March was also observed within this pipeline as well as at the source. The subsequent sampling visit revealed an increase in bacterial numbers reaching the Farpoint site. The increase in bacteria in the source water concentration was not observed at the Taylorside/Ethelton pipeline. This lack of observation is attributed to the timing of the site visits and thus increased levels were only observed downstream within the network itself. No peaks were observed in the Lucky Lake North pipeline 2000 data, likely a function of inexperience with the method during the early stages of the project. Particulate matter in the sample made it difficult to properly stain and enumerate the bacteria. Particles can interfere with the enumeration as the stain coats the particles when it should be staining bacteria. As a result of bacteria's affinity for particulate, the Midpoint site enumerations were quite variable.

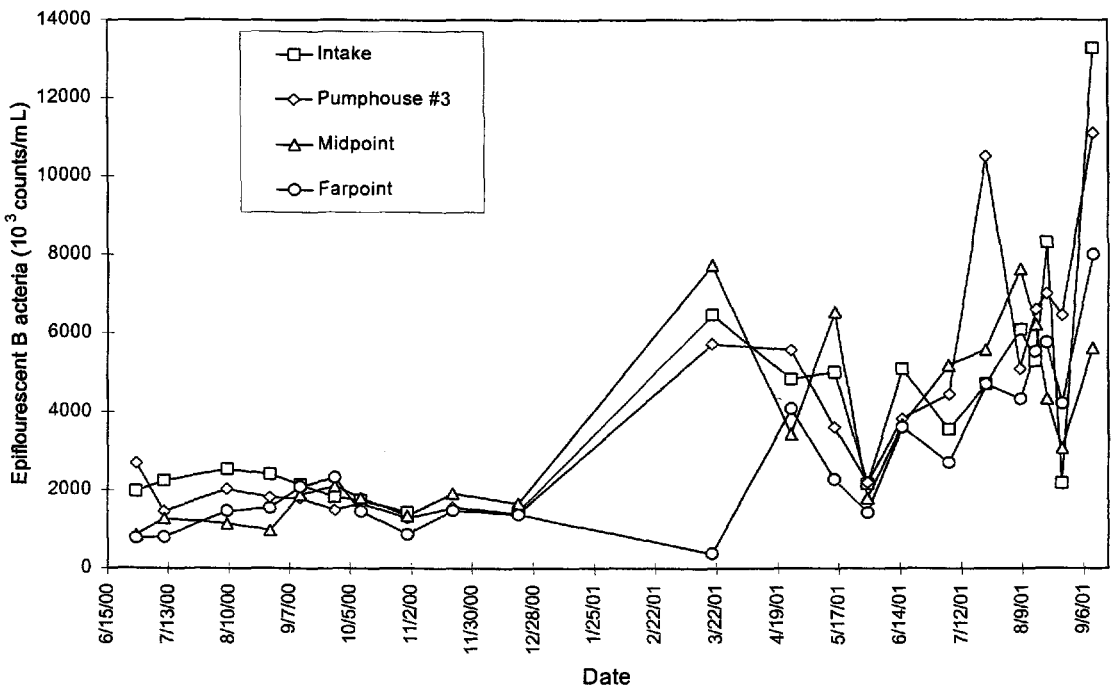

Figure 6.17 Epifluorescent bacteria counts recorded from the Lucky Lake North monitoring sites.

6.2.6 Particle Size Analysis

Particle counts for 2001, after re-calibration of the analyzer, are presented in Figures 6.18 and 6.19. The 2-5 and 5-10 μm ranges show significant concentrations of particles at the intake and booster station #3 for the early spring and summer months. This may have been due to runoff into the bay in which the intake was located, or a result of wave action combined with the low reservoir level. The particle counts appear to decrease as the water reaches the comparatively quiescent North branch pipeline.

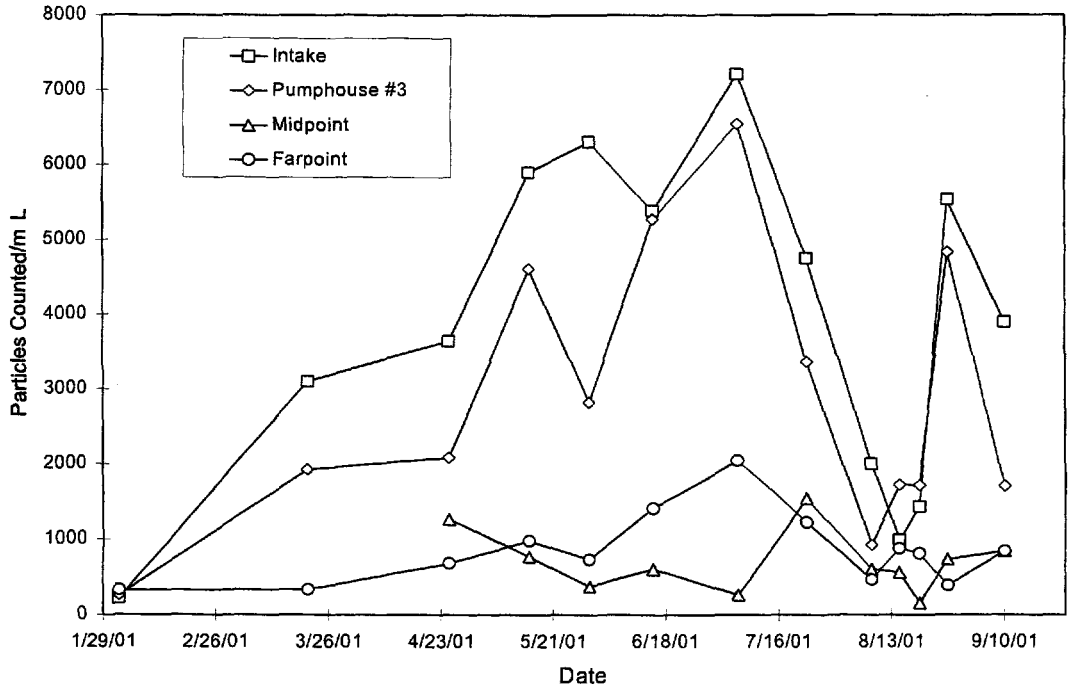

Figure 6.18 Particle counts for 2-5 μm particles as recorded at the Lucky Lake North monitoring sites.

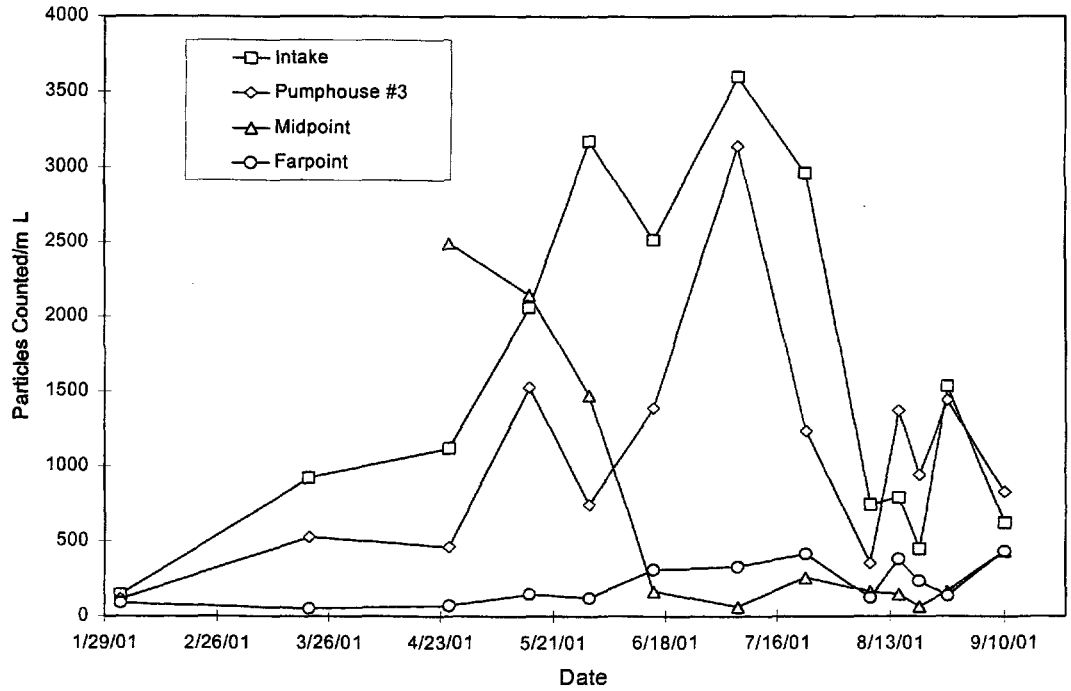

Figure 6.19 Particle counts for 5-10 μm particles as recorded at the Lucky Lake North monitoring sites.

Figures 6.20 and 6.21 show the particle size distribution at the two user sites. The two sites are somewhat different in the particle size which dominates the distribution. The Farpoint typically showed the majority of particles were finer than those observed at the Midpoint. This may have been a result of sedimentation of the larger particles in transit to the Farpoint.

The inflow rate to the Midpoint user cistern was generally very low due to the method of actuation, causing a low velocity in the service line that promoted deposition of particles. Attempts were made to control the flow rate during sampling, but the high pressure at the Midpoint made it difficult to control the sampling flow rate without stirring up sediment in the lines. The Midpoint samples appeared to carry larger particles as a result of this sampling velocity. The particle counts are influenced by sampling velocity and thus peak particle count values resulting from high rate sampling are not an accurate reflection of the particulate matter in the water delivered to the Midpoint site.

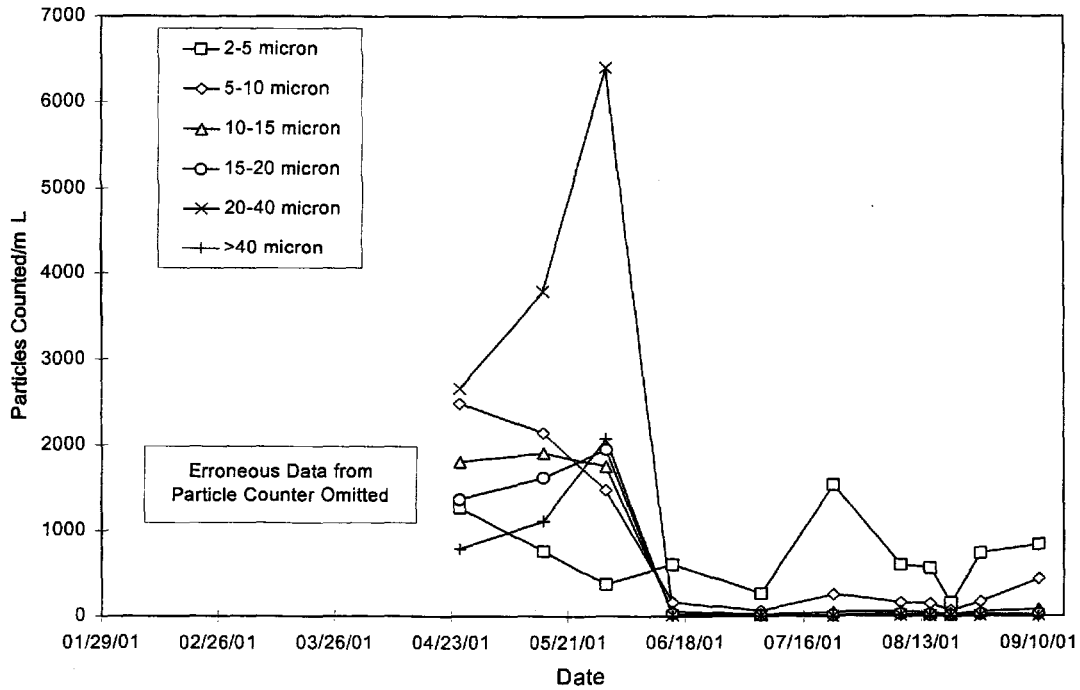

Figure 6.20 Particle size distribution recorded from the Lucky Lake North Midpoint monitoring site.

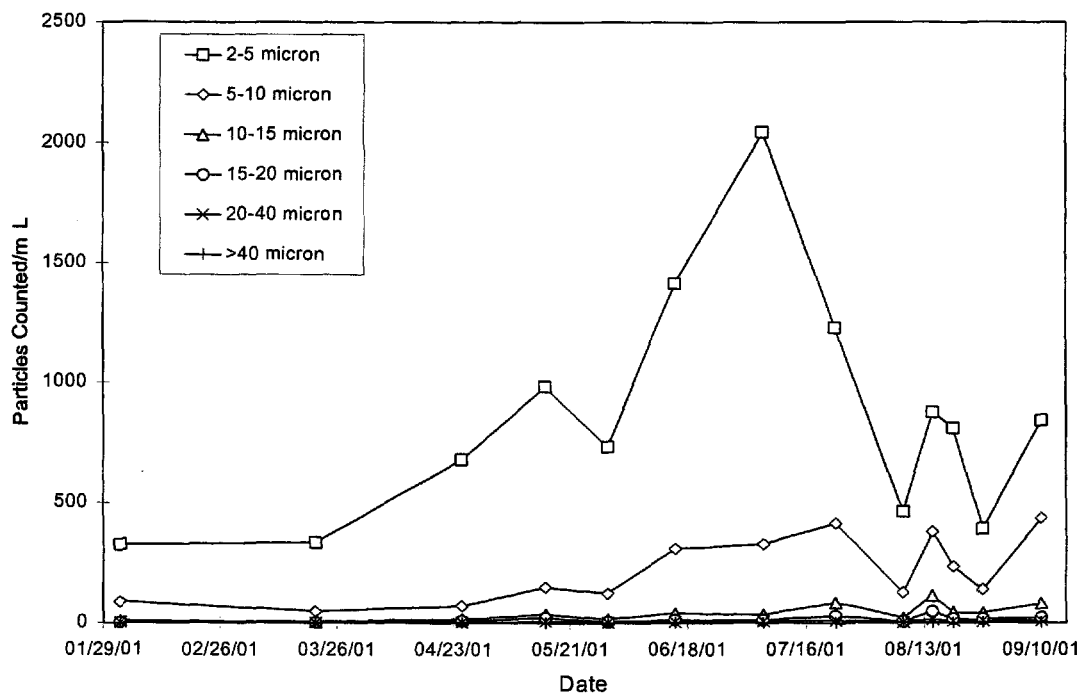

Figure 6.21 Particle size distribution recorded from the Lucky Lake North Farpoint monitoring site.

6.3 Discussion of Water Quality Results

6.3.1 Interpretation of Data

LeChevallier (1990) states:

"Temperature is perhaps the most important rate-controlling parameter in the microbial growth process. Temperature affects, directly or indirectly, all of the factors that govern the microbial growth."

Temperature is a parameter that has a strong influence on water quality for the two rural water distribution networks studied. Peak values of DOC, chlorine decay, turbidity, and epifluorescent counts were observed to coincide with peak temperatures in the bulk water of these systems. Conversely, many of the parameters showed the lowest concentrations or indicated more stability when bulk water temperatures were low. Temperature is a controlling factor in distribution system water quality because it affects the rates of reaction between organic matter, chlorine residuals and bacteria (LeChevallier, 1990). The variability of the parameter values during peak temperatures is believed to be the result of an increased rate of interaction of the parameters owing to the temperature dependency of reaction rates. LeChevallier (1990) notes that of the factors affecting water quality, temperature is the most difficult to control, as forcing a change in temperature is limited by the feasibility of doing so. The cooling effect of ground temperature appears to be a very positive influence in the case of rural water distribution systems.

Rainfall has been noted to influence the concentrations of organic carbon in source water reservoirs through runoff (Volk and LeChevallier, 2000; Gatel et al., 1995; Liu et al., 2002; Escobar and Randall, 1999a; Niquette et al., 2001), but a conclusive correlation could not be determined during this study as rainfall and weather data over the drainage basin were not analyzed. The duration of high DOC concentrations in the Taylorside/ Ethelton pipeline, prolonged by the flood wave which followed the spike of DOC from the alternate source (see Section 6.1.2), suggests that there is a relationship between DOC concentrations and runoff into water bodies. Until the spike of DOC, the DOC concentrations in the Taylorside/Ethelton pipeline had been relatively low. The Lucky Lake North branch also showed relatively low DOC values throughout the

summer of 2001 (see Figure 6.14). This may have been due to a lack of precipitation runoff from the surrounding land and small runoff surges into the reservoirs.

Elevated levels of DOC provide substrate for bacteria in the distribution system which can multiply at higher rates with increased temperatures. DOC concentrations were noted to change throughout the pipelines (see Figures 6.2 and 6.14), which can be an indicator of biological uptake or reaction with free chlorine (in the case of the Taylorside/Ethelton pipeline). DOC concentrations during periods of low temperature were found to vary little throughout the pipelines indicating a lack of interaction or slowed reaction with bacteria or free chlorine.

The BDOC analyses showed a very small portion of the DOC is available for bacterial growth. The observed BDOC concentrations rarely exceeded the biofilm threshold value of 0.2 mg/l, with the exception of increased levels coinciding with late summer (see Figures 6.3 and 6.15). Initially it was thought that the low BDOC readings may be the result too few inoculum bacteria. Bacteria in a sample of river water were enumerated using the epifluorescent technique and were found to be present in sufficient number for use as inoculum. Secondly, it was theorized that the low reading could be the result of well nourished river bacteria being unable to adapt to the low substrate environment of the treated Taylorside/Ethelton water. This theory was disproved by the variable results observed in the Lucky Lake North branch samples, which are raw waters from the same river as the inoculum bacteria. The conclusion was made that the biodegradable concentration of organic carbon was typically below the 0.2 mg/L limit of applicability for the method of Servais et al. (1989).

Bacterial populations typically peaked in the late summer and early spring (see Figures 6.5 and 6.17). Van Der Wende (1989) suggests that suspended bacteria are a result of biofilm growth within a network, particularly in treated water distribution systems carrying a chlorine residual, which have a low number of seed bacteria. This is not necessarily the case in untreated distribution networks such as the Lucky Lake North branch. While biofilms likely contribute a large number of bacteria to the bulk water concentrations in the Lucky Lake North pipeline, seed bacteria from the reservoir likely supplement the suspended concentration.

In the case of the Taylorside/Ethelton pipeline, when a free chlorine residual was maintained, the number of viable bacteria (HPC) were below the limit of detection (see Figure 6.8). High temperatures increase the rate of reaction of chlorine with organic matter present in the bulk water, depleting residuals. Concurrent inactivation of biomass in the network would also increase the rate of decay of chlorine. Once chlorine residual concentrations were depleted in the Taylorside/Ethelton pipeline extremities (Farpoint) an increase in the number of viable cells was recorded. The maximum recorded concentration of viable bacteria remained below acceptable limits. The substrate (see Figure 6.3) and free chlorine (see Figure 6.8) values were at levels suspected of promoting re-growth, but temperature at the Farpoint location was only around 8 °C (see Figure 6.1) which is believed to have limited the multiplication of the organisms. In the Lucky Lake North branch where chlorine is not used, bacterial activity increased at higher temperatures (refer to Figures 6.13 And 6.17). The growth of bacteria during the peak temperature season was most pronounced at the intake and booster station # 3 where temperatures were the highest, often above the biofilm threshold level of 15 °C.

The lowest, and most variable chlorine residual concentrations were observed during peak temperatures (see Figures 6.6 and 6.7), suggesting high rates of reaction with the elevated levels of organic matter and bacteria present. When networks experience factors that contribute to decay of chlorine residuals below acceptable levels, flushing the mains has been shown to improve residual concentrations by bringing in water with higher chlorine concentrations from the upstream sections of the network (Gatel et al., 1998). This is only a temporary solution but due to the temporary nature of source water problems and seasonal increases in decay rates, flushing may be a feasible solution to the low chlorine residual issues in rural networks.

The general trend appears to be that if input DOC decreases, chlorine residuals remain relatively constant and bacterial counts are low. If the input DOC increases, chlorine residuals decrease (which in some cases corresponded to an increase in BDOC) and bacterial counts increase.

6.3.2 Spike Input of Poor Quality Source Water

On August 7th, 2001 the water treatment plant in Melfort had to switch to an alternate source of raw water due to a break in the raw water supply line from the Codette reservoir. The alternate source carried extremely high concentrations of DOC into the water treatment plant for several days. The DOC recorded from a sample taken from the water treatment plant effluent on August 9th was 11.3 mg/L. Measurements within the branch network on the same day indicated that this high concentration of DOC had not yet reached the Taylorside/Ethelton pipeline. The introduction of the huge DOC concentration consumed chlorine rapidly. The free chlorine residuals leaving the water treatment plant were reduced from an average of 1.23 mg/L on August 7th to a level of 0.97 mg/L on August 8. The water treatment plant residuals were increased, and varied from approximately 1.23 to 1.58 mg/L over the following five days.

The bacterial samples taken at the water treatment plant on August 9th also showed an increase in the number of organisms. During the following site visit on August 13th, the DOC and bacteria concentrations had increased, and chlorine residuals had decreased throughout the branch network. Figure 6.22 shows the changes in values between the water treatment plant and the Farpoint site in subsequent visits.

Based on the results of the computer modelling (discussed in Chapter 7), the average residence time to the Farpoint in this system during this period was just over one week. As the high organic content water moved through the system, chlorine residuals were consumed in reaction with the DOC, producing higher levels of BDOC. The higher substrate concentration and depleted chlorine residuals allowed an increase in the number of organisms. This is evident looking at the Farpoint concentrations between the August 13th and August 21st in Figure 6.22. The peak bacteria concentration was recorded at the Farpoint on August 21st. This increase in bacterial concentration coincided with a reduction in BDOC, which is indicative of consumption. The water treatment plant effluent DOC concentrations decreased between August 8th and August 13th.

The water treatment plant returned to the primary raw water source on August 16th, just as a flood wave from a large storm in Alberta reached the Codette reservoir. The DOC concentration after treatment of this poor quality, highly turbid raw water, reached

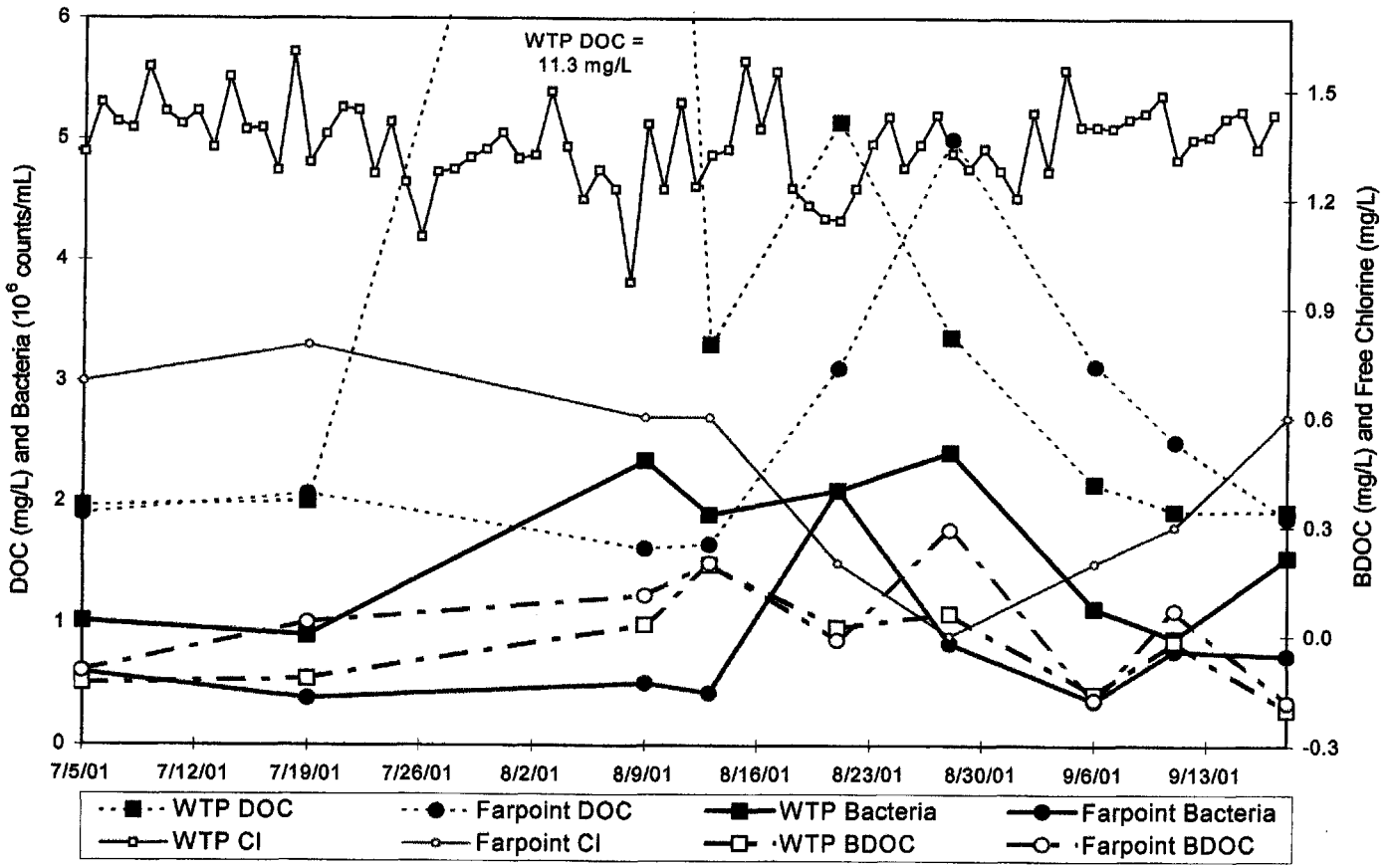

Figure 6.22 Variation of parameters between water treatment plant and Farpoint site on Taylorside/Ethelton pipeline as a result of a change in source water.

5.3 mg/L on August 21st. The chlorine residual at the Farpoint was very low during the same visit. Residual free chlorine observed at the user sites upstream during the same site visit were even lower.

Interestingly, the first DOC spike was not observed at the Farpoint site, although increased levels were noted elsewhere in the branch network. Data from following visits are more difficult to explain. The introduction of the second DOC spike into the system (from the primary source) worked through the network. The chlorine residual at the Farpoint the following week (August 28th) reached a concentration of zero and HPC organisms were detected. During this visit the DOC and BDOC increased to the highest levels at the Farpoint, while bacteria counts decreased. One would expect the following visit to have reported the highest counts of bacteria at the Farpoint, but this was not the case. The bacteria count had decreased further. DOC and BDOC decreased while the chlorine residual increased marginally.

In an attempt to explain these observations, several theories were developed based on the mechanisms of interaction reported for other systems. The theories are highly speculative. Without the benefit of high resolution sampling to describe the complex interaction of these parameters during this event, there cannot be any confidence in an explanation of this occurrence.

6.4 Biofilm Sampling

Sections of the two rural water pipelines studied were removed and analyzed for biofilm density in August of 2001. The following subsections present the findings of the analysis.

6.4.1 Taylorside/Ethelton Pipeline Results

Two sites were selected on the basis of extended residence times indicated by low water use. The Jellicoe farm is located between the Booster Station and the Midpoint site (see Figure 3.1). The Groat farm is at the end of a lengthy service connection east of the Farpoint site (see Figure 3.1). The feed water to the Groat farm comes from a section of line between the Midpoint and Farpoint sites. Figure 6.23 shows the pipe samples taken

from these two sites. The pipe samples appeared to have a mineral film adhered to the interior, consistent with the observations of Ridgway and Olson (1981). This film was less obvious at the Groat farm. The condition of the wetted pipe surface was otherwise normal in appearance.

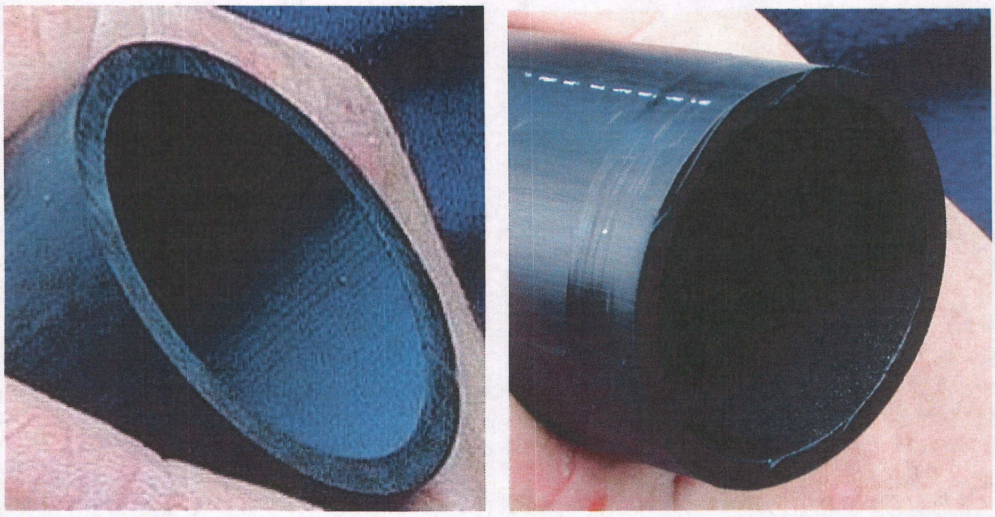

Figure 6.23 Pipe samples taken from the Taylorside/Ethelton Pipeline at the Jellicoe farm (left) and the Groat farm (right). Service line shown is 41 mm diameter.

Table 6.1 shows the results of the biofilm density analyses. The biofilm density was low and consistent between the two sites. The viable (HPC) and total (epifluorescent) bacteria were enumerated. The biofilm density is the number of bacteria per unit area of the pipe sample interior surface. The maximum potential count represents the increase in bacterial concentration in the flowing water if all of the organisms were simultaneously released from the pipe surface into the flowing water. The major bacterial species identified from the sonicated (see Section 4.3.5.2) samples is also shown. The major species identified in the biofilm analysis (*Bacillus Megaterium*) commonly occurs in soil and water.

Table 6.1 Taylorside/Ethelton biofilm analysis results.

Parameter	Jellicoe Farm	Groat Farm
Calculated Biofilm Density:		
HPC CFU[(1)]/cm^2	45	36
Epifluorescent Organisms × 10^3/cm^2	1432	2146
Maximum Potential Count:		
HPC CFU[(1)]/ml	43	35
Epifluorescent Organisms × 10^3/ml	1392	2086
Major Species Identified	*Bacillus Megaterium*	*Bacillus Megaterium*

[(1)] Colony forming units.

6.4.2 Lucky Lake North Branch Pipeline Results

Owing to the low number of users on the Lucky lake North line and the access permitted by the residents of the Midpoint and Farpoint sites, these two locations were chosen for biofilm analysis. Figure 6.24 shows the samples taken from these sites. Each pipe sample exhibited a film adhered to the pipe interior surface. The thickness and visibility of the film was much greater than those observed on the Taylorside/Ethelton samples. The colour and slimy nature of the film on these samples led to the belief that there might be a substantial concentration of attached bacteria.

The results of the biofilm density analysis is shown in Table 6.2. A two order of magnitude increase in HPC bacteria concentrations over the Taylorside/ Ethelton samples was noted. The major species identified in the analyses were *Pseudomonas Chlororaphis* and *Pseudomonas Putida*. Both are commonly found in surface waters. *Pseudomonas Putida* is believed to be an opportunistic pathogen (Momba et al, 2000).

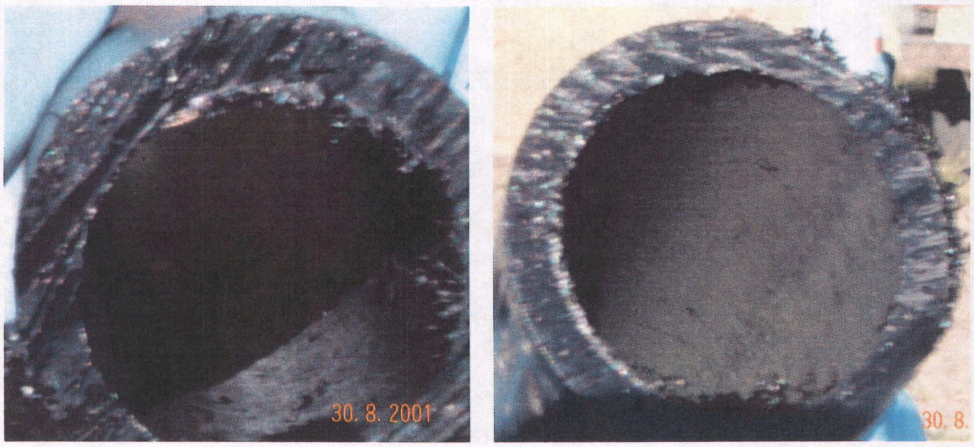

Figure 6.24 Pipe samples taken from the Lucky Lake North branch pipeline at the Midpoint (left) and the Farpoint (right). Service line shown is 41 mm diameter.

Table 6.2 Lucky Lake North Branch biofilm analysis results.

Parameter	Midpoint	Farpoint
Calculated Biofilm Density:		
HPC CFU[(1)]/cm^2	2370	2510
Epifluorescent Organisms × 10^3/cm^2	6405	8849
Maximum Potential Count:		
HPC CFU[(1)]/ml	2530	2540
Epifluorescent Organisms × 10^3/ml	6834	8693
Major Species Identified	*Pseudomonas Chlororaphis*	*Pseudomonas Putida*

[(1)] Colony forming units

6.4.3 Discussion of Biofilm Analysis Results

The samples taken from the Taylorside/Ethelton pipeline did not indicate significant biofilm development at the two sites. Viable organism density was extremely low at these locations indicating that conditions were not ideal for the development of biofilm. The low concentrations of bacteria present suggest that *Bacillus Megaterium*

may have been detected as a result of incomplete cleaning and disinfection of the exterior surface of the pipe sample. The limited viable numbers (HPC counts) and large numbers of inactivated bacteria (Epifluorescent counts less HPC counts) recovered at these sites suggest that there may be a nutrient limitation caused by nutrient consumption during the stagnant periods characteristic of the service lines. The influx of new water during a cistern activation may also carry higher chlorine residuals which neutralize any attached bacteria in these laterals.

In contrast, the Lucky Lake North Branch pipeline samples showed a higher number of viable organisms, likely owing to the higher temperatures, ample DOC, inoculum bacteria and a lack of a chlorine residual. Despite these factors the viable biofilm concentrations remain lower than expected for a raw water distribution network. Temperatures increase above the biofilm threshold value of 15 °C near the head of the of the network for a portion of the year. Within the branch network, the values recorded did not exceed 14 °C. The number of viable organisms, if released into the water column, may represent a significant concentration but as suspended viable organisms were not enumerated for the Lucky Lake North pipeline it is not known if this would cause a significant increase in the viable suspended population. Based on observations of Momba et al. (2000), 1000 viable organisms are present in a biofilm for every one detected in the bulk water column. The sudden release of these bacteria would cause a drastic increase in the number of bacteria transported through the system.

The viable numbers recovered from the Taylorside/Ethelton samples are much lower than some of the published densities. LeChevallier et al. (1987) presented a summary of biofilm densities taken from pipe samples in treated water distribution networks of various age. The number of viable organisms presented in LeChevallier et al. (1987) were enumerated by HPC analyses. The two pipes most similar in age to the rural water pipelines studied had a density of 44,000 CFU/cm^2 for an 8 year old cement lined ductile iron pipe and a density of 800 CFU/cm^2 for a 9 year old asbestos cement main. Both of the mains were located in New Jersey, USA. It is remarkable that the biofilm densities observed in the Lucky Lake North pipeline, which distributes untreated water without residual free chlorine, are lower than some of those in the treated water

distribution systems reported by LeChevallier et al. (1987). Some of the factors causing the differences in biofilm densities between this study and the published values are likely pipe material, the method of disinfection employed, and the bulk water temperature. This comparison indicates the biofilm densities in the Taylorside/Ethelton and Lucky Lake North branch pipelines are marginal compared to the potential biofilm density in systems located in warmer climates. As a point of interest, the highest biofilm density presented in the publication by LeChevallier et al. (1987) was 2.36×10^6 CFU/cm^2 for a 90 year old cast iron main.

Ridgway and Olson (1981) reported that the attached biofilm populations recovered from a main taken from an urban distribution system were inconsistently distributed throughout the pipe sample, often patchy in appearance. Therefore, the samples taken in this rural pipeline may not have coincided with a location of an attached population. A difference between the total bacteria enumerated in replicate samples taken from the Groat farm was evident, suggesting that the intermittent nature of biofilm development is a distinct possibility in the Taylorside/Ethelton pipeline. Regardless of what interactions may be occurring in these lengthy service lines, the continually low temperatures present in the two systems studied appear to effectively inhibit biofilm formation.

In retrospect, an area of the pipeline experiencing limited residence times (and higher substrate concentrations) may have been more likely to support the development of a biofilm, even in the presence of higher chlorine residuals.

7.0 HYDRAULIC MODELLING AND CHLORINE DECAY ANALYSIS

This chapter describes the construction and output of the model created for the simulation of hydraulic residence times in the Taylorside/Ethelton pipeline, and the results of the chlorine decay analysis.

7.1 Hydraulic Model

7.1.1 Model Construction

Data related to the construction of the Taylorside/Ethelton pipeline were obtained from a GIS map created and provided by PFRA. The pipe lengths and diameters could be queried from the GIS map. The pipeline layout, lengths, and diameters were then created in EPANET. Construction details of the regional pipeline supplying the branch network were provided by the Saskatchewan Water Corporation in AutoCAD format. These details were incorporated into the model in a similar fashion. The make and model numbers of the booster station pumps and pneumatic tanks were recorded and details were obtained from the manufacturer's literature.

General purpose and flow control valves were used within EPANET to simulate the metering and solenoid control assemblies at the user sites. Cisterns were modelled using tanks. The configuration used to model each of the 44 user sites is shown in Figure 7.1. Cistern dimensions and volumes for each of the users were provided by the regional pipeline association. Operating volumes for the modelled cisterns were based on observations at the Midpoint and Farpoint sites. Operating volumes were 30% of the total cistern volume for farming operations and 50 % of the total volume for livestock operations. Livestock operations were identified by a notable increase in usage reported in the billing records in the first two quarters of 2001; others were assumed to be regular

farming operations. The flow versus pressure head relationship through the service connection valves and meters was determined by observation of the flow rates and line pressure at the two monitored user sites. To simulate the headloss through the connection valves and meters, the curve of flow and pressure head observed in the field was programmed as a general purpose valve (GPV) in EPANET upstream of the cistern. An example of the flow and pressure head data from the Farpoint site used to derive this relationship is shown in Figure 7.2. Flow into the service line was allowed by opening and closing the flow control valve, based on tank level. This simulated the action of the

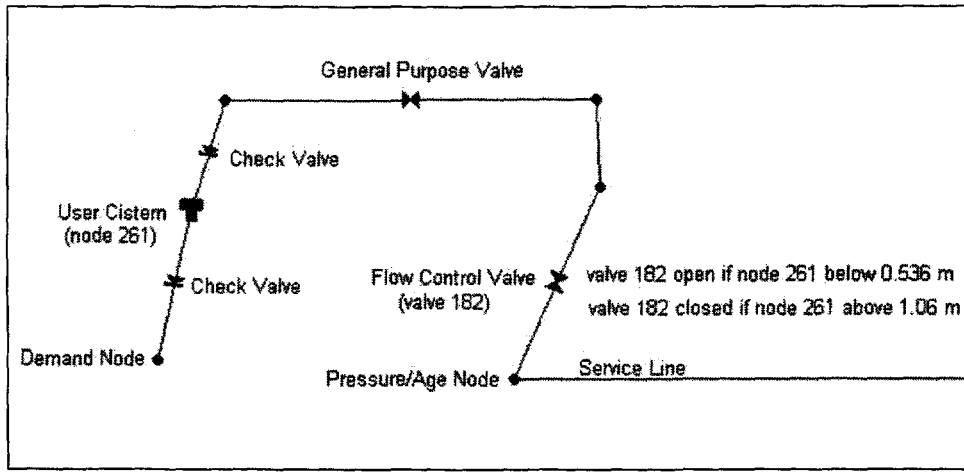

Figure 7.1 EPANET model representation of user connections.

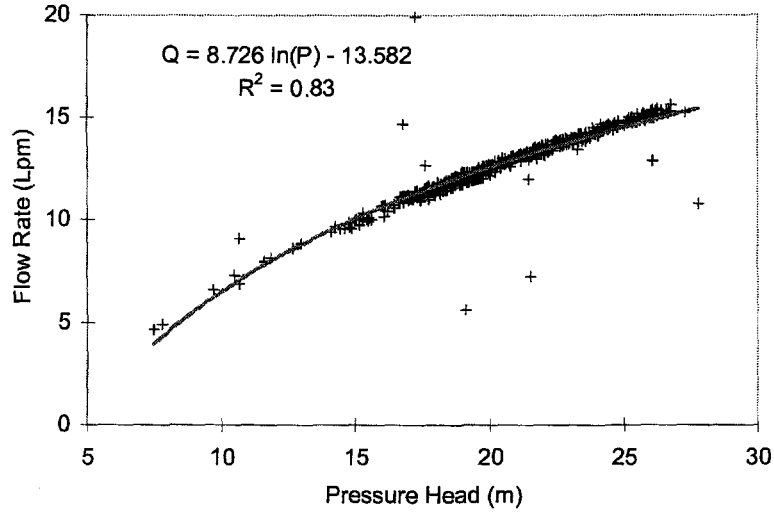

Figure 7.2 Equation used for general purpose valve in EPANET.

solenoid valve and float assemblies in the field. The flow control valve also served as a pressure reducing valve (PRV). The PRV would close partially or fully when the downstream pressure head exceeded the set point of 28 m (276 kPa). If the upstream pressure was less than the set point, the valve would remain fully open.

Simple control statements were written in EPANET instructing the flow control valve to open when the tank level dropped below the fill level and to close when the tank level rose above the full level. These control statements modelled the behaviour of the low and high level floats present in the user's cisterns. The control statements used at the modelled Midpoint site are shown in Figure 7.1. Check valves were placed in the model to stop siphoning from the tank back into the network, in effect modelling the air gap required in the real systems.

User demand was simulated by drawing water from the tank at the demand node. The magnitude of the user demand varied according to a demand pattern that was synthesized based upon observations at the booster station throughout the quarter modelled. Initial estimates of user demand were assigned to the nodes based on billing records from the pipeline association. The modelled and observed volumes passing through the booster station were found to differ, which led to the discovery that the billing records from the pipeline association were not the same as the metered volumes observed at the two user sites. Quarterly readings from the Saskatchewan Water Corporation meter at the head of the network were used to calibrate the model. This was done so that the modelled Taylorside/Ethelton system's total quarterly demand closely resembled the records from the Saskatchewan Water Corporation. The readings from the Saskatchewan Water Corporation were considered more accurate than the billing records as the billing records were based on volumes reported by the users.

The Taylorside/Ethelton booster station is equipped with one centrifugal end-suction pump and two hydro-pneumatic tanks, off-stream, in parallel with one another. The purpose of the hydro-pneumatic tanks is to prevent rapid pump cycling. The EPANET model of the Booster Station is shown in Figure 7.3. Within EPANET these two tanks were modelled based on the high and low pressure settings for the pump. The two hydro-pneumatic tanks were simulated by creating one standard tank in the model

with a water surface open to the atmosphere. The full and empty water surface elevations of the standard tank were programmed such that the pressure heads at these elevations were equal to the high and low pressure set points for the pump. The storage volumes of the hydro-pneumatic tanks at the pump set point pressures were taken from manufacturer's literature. The diameter of the modelled tank was programmed such that the volume of water between the high and low levels equaled the storage of the hydro-pneumatic tanks. The pump was turned on and off with rule-based controls dependent on the pressure head that the water surface in the modelled tank exerted on a downstream node (see Figure 7.3). The pump curve was obtained from the manufacturer's literature and entered into the model.

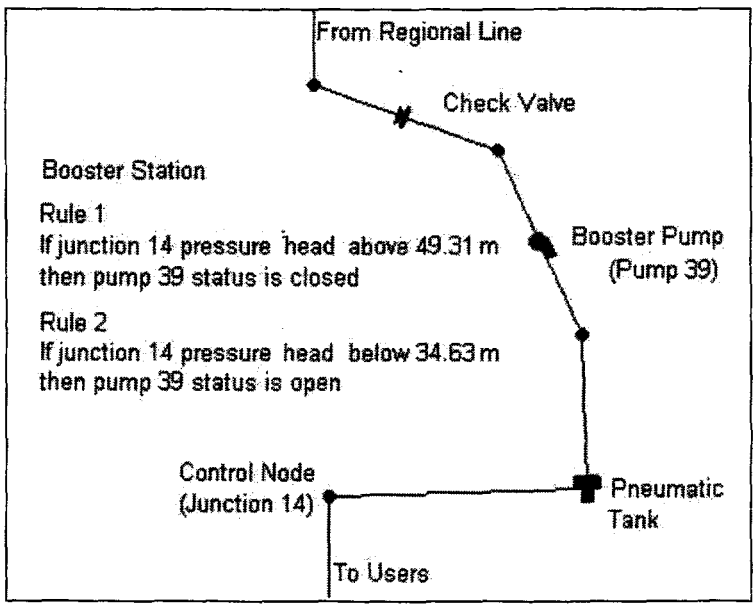

Figure 7.3 EPANET model representation of the Taylorside/Ethelton booster station.

The regional pipeline was modelled using physical data obtained from the Saskatchewan Water Corporation. Figure 7.4 shows the model layout. In the interest of showing the branch network at scale and also the regional line within the viewport, the regional line was not visually represented to scale. However, accurate lengths of the sections of the regional main used for hydraulic calculations were manually entered into the attributes table for each pipe section. This allowed the entire system to be viewed at a

scale appropriate for observing the network reaction, while maintaining the actual characteristics of the regional line for calculation purposes.

Other branch lines supplied by the regional line upstream of the Taylorside/ Ethelton branch were modelled as demands from nodes along the regional line. Network demands for these laterals were determined from Melfort Rural Pipeline Association (MRPA) records. Usage patterns were assumed to be similar to those observed at the Taylorside/Ethelton booster station therefore the synthesized demand patterns for each quarter used for the modelled Taylorside/Ethelton network were also applied to these branch lines.

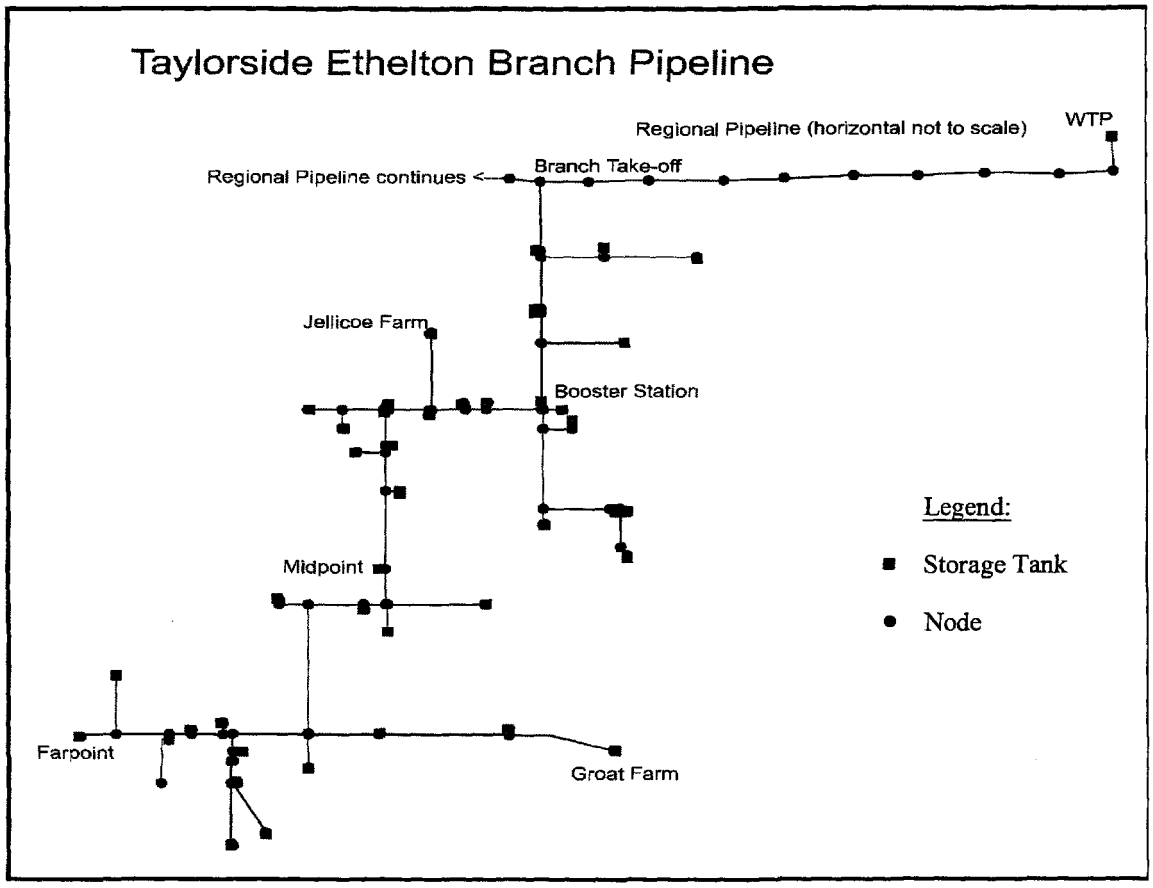

Figure 7.4 EPANET model representation of the regional pipeline and Taylorside/ Ethelton branch network.

Flow in the regional line downstream of the Taylorside/Ethelton pipeline supply point was modelled as one demand. The same demand pattern applied to the branch lines was applied to the downstream demand.

Water supplied from the Melfort water treatment plant was modelled as a constant head reservoir of unlimited supply. The reservoir elevation was adjusted in the first quarter to provide the correct inlet pressure at the booster station and was assumed to remain constant throughout the remainder of the quarters modelled.

Nodes in the network were placed to allow representation of changes in pipe diameter or material, bends, withdrawal of water (demands), and changes in elevation (i.e. local high or low points). Elevations for the regional line were taken from the AutoCAD drawings supplied by the Saskatchewan Water Corporation. The elevations of the Taylorside/Ethelton branch network were taken from surface contours provided by PFRA and adjusted to an assumed 3 m burial.

7.1.2 Flow and Pressure Simulations in EPANET

Flow and pressure simulations were conducted for each quarter studied. Most of the user cisterns were set at the full level for the beginning of the simulation. This meant that an initial 'run up' period of adjustment was required for all of the users in the system to develop a pattern of cistern filling from the network. As the number of times the users withdraw water from the network increased, the flow and pressure patterns developed in the model more closely simulated the random demand and flow characteristics of the actual network. The model simulation time was only set up to be half of the quarter as the model output would become somewhat cyclic, due to the repetition of the daily demand patterns, following the run up period. This run up period was deemed to be complete when the maximum residence times were reached at the system peripherals. In this case, the Groat farm was used as the indicator site because it consistently had the longest residence times in the system (see Figure 7.10). A comparison of modelled and observed pressure head, and modelled and observed flow rate at 15 minute intervals at the booster station is shown in Figure 7.5.

In Figure 7.5 the range of flow rate and pressure head fluctuations differs between the modelled and observed parameters. The data reported in the model is an instantaneous value whereas the observed values are an average of readings taken over a 15 minute time interval. This difference in variation magnitude is present in all of the examples shown. The modelled inlet pressure head is also affected by the calculation of the flow into the pump by the model, which is why it appears as a negative value at some points. Figures 7.6 and 7.7 show the modelled and observed flow rate and pressure head at the two user sites over the same interval as Figure 7.5.

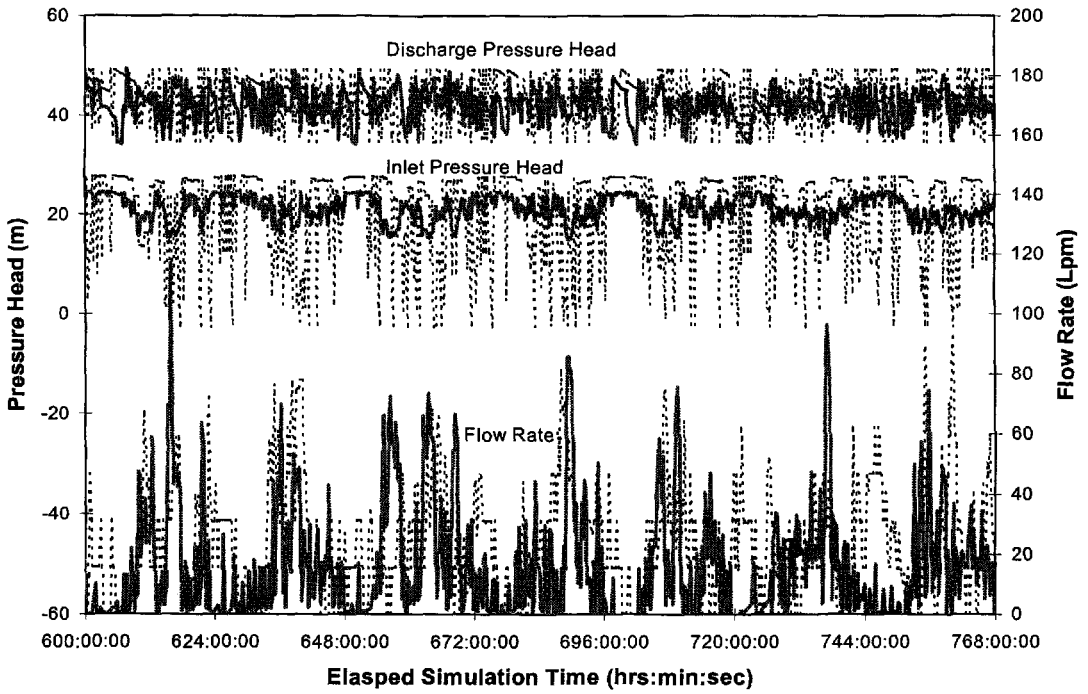

Figure 7.5 Example of modelled (dashed) and observed (solid) parameters at the Taylorside/Ethelton Booster Station for one week in the 3rd quarter of 2000.

In Figure 7.6, the number of observed cistern fills is less than the modelled number of cistern fills. A check of the volumes produced in the simulation compared to the actual use indicated that the difference shown in Figure 7.6 is because demand by the Midpoint user increased toward the end of the observed quarter. As the modelled cistern fills are on a regularly repeating cycle due to the use of an average demand and repeating pattern, differences in the frequency between modelled and observed cistern fills can be

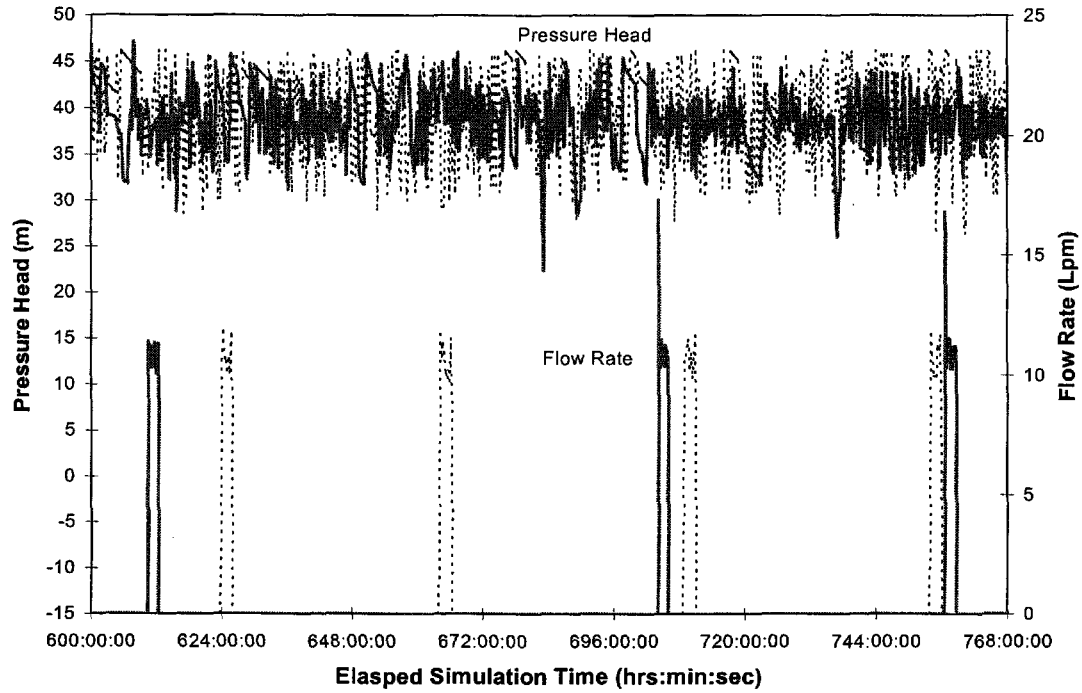

Figure 7.6 Example of modelled (dashed) and observed (solid) parameters at the Taylorside/Ethelton Midpoint user for one week in the 3rd quarter of 2000.

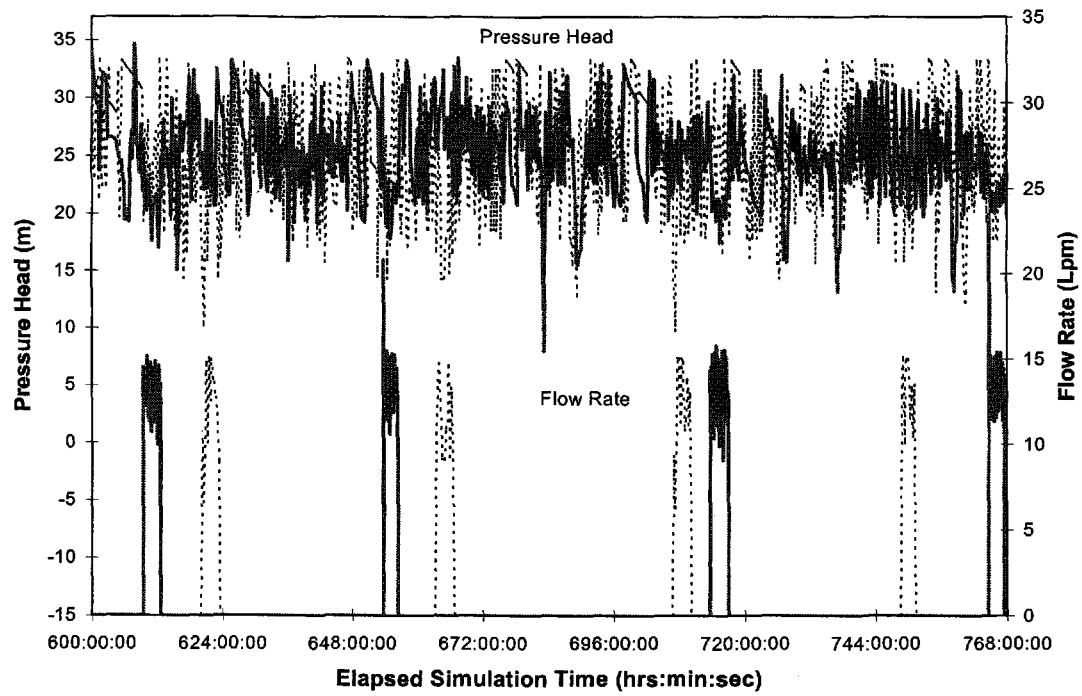

Figure 7.7 Example of modelled (dashed) and observed (solid) parameters at the Taylorside/Ethelton Farpoint user for one week in the 3rd quarter of 2000.

expected. The total volume supplied in the model is equal to the observed volume over the period of simulation. Comparison of modelled and observed flow at the Farpoint user (shown in Figure 7.7) also indicated a close match.

Table 7.1 lists the recorded and modelled quarterly system flow volumes. This water balance is within acceptable limits according to the criteria specified in the publication *Advanced Water Distribution Modelling and Management* (Haestad Methods et al., 2003), with the exception of the 1st quarter of 2001. Haestad Methods distributes the commercially available modelling software WaterCAD. The negative values in Table 7.1 indicate the modelled volume was less than the observed volume.

Table 7.1 Percent difference between observed and modelled flow volume in the Taylorside/Ethelton pipeline for each quarterly reporting period modelled.

Quarter Modelled	Booster Station	Midpoint	Farpoint
3-2000	-2.07%(1)	-3.92%	-2.9%
4-2000	-3.26%	-1.9%	-2.48%
1-2001	-5.95%(2)	-1.48%	+0.02%
2-2001	-3.25%	-0.34%	-4.02%
3-2001	-1.85%	-2.66%	-2.50%

(1) Negative values indicate the modelled volume is less than the observed volume.
(2) Marginally exceeds error criteria specified by Haestad et al. (2003).

As is evident by visual inspection of Figures 7.5 to 7.7, the modelled and observed system average pressure heads also closely agree. Comparison of the values illustrated in these figures is presented in Table 7.2.

Table 7.2 Comparison of observed and modelled flow rate and pressure head at the Taylorside/Ethelton monitoring sites for the simulation shown in Figures 7.5 to 7.7.

Site and Parameter	Observed Average	Observed Standard Deviation	Modelled Average	Modelled Standard Deviation
Booster Station Inlet Pressure Head (m)	20.59	2.97	19.28	9.69
Booster Station Discharge Pressure Head (m)	42.33	3.52	42.8	5.01
Booster Station Flow Rate (Lpm)	24.95	25.06	23.95	18.68
Midpoint Pressure Head (m)	38.78	3.69	38.91	5.09
Midpoint Flow Rate (Lpm)	10.84	1.00	10.80	0.67
Midpoint Pressure Head (m)	25.29	5.18	25.11	5.35
Midpoint Pressure Head (m)	12.96	2.21	12.10	2.22

7.1.3 HRT Simulations

Modelling of HRT proved to be an arduous task. Representation of the hydro-pneumatic tank present at the pumphouse caused problems with the simulations depending on the algorithm selected for tracking water age in the modelled tank within EPANET. Under some simulations the age of the water would reset to zero whenever the tank was emptied. A water age equal to zero exiting the tank is unreasonable given the age of the water being fed to the tank. To offset this error, the minimum level of the tank was adjusted in EPANET to just above the tank bottom. The parcel of modelled water in the bottom of the tank would continue to age and would mix with the incoming fresh water during the course of the simulation. This error produced increasing fluctuations of age downstream, effectively increasing the average age with time.

The errors described above were reduced by using a 'last in first out' algorithm coupled with a small minimum volume. The resulting age downstream of the booster station was affected by periodic circulation of stagnant water in the bottom of the tank

which caused some spikes in water age. These effects can be seen in Figure 7.8. In comparing the average water age upstream and downstream of the tank, the effect that these age spikes had on the average residence time was found to be minimal. For the example shown in Figure 7.8, the average age before the tank was found to be 48.8 hours whereas the downstream average age was 50.6 hours. This represents a change of 3.6% in the average age at that location. With higher HRTs, such as those at the Midpoint and Farpoint, this error is reduced to 1.7 and 1.1% of the average age, respectively.

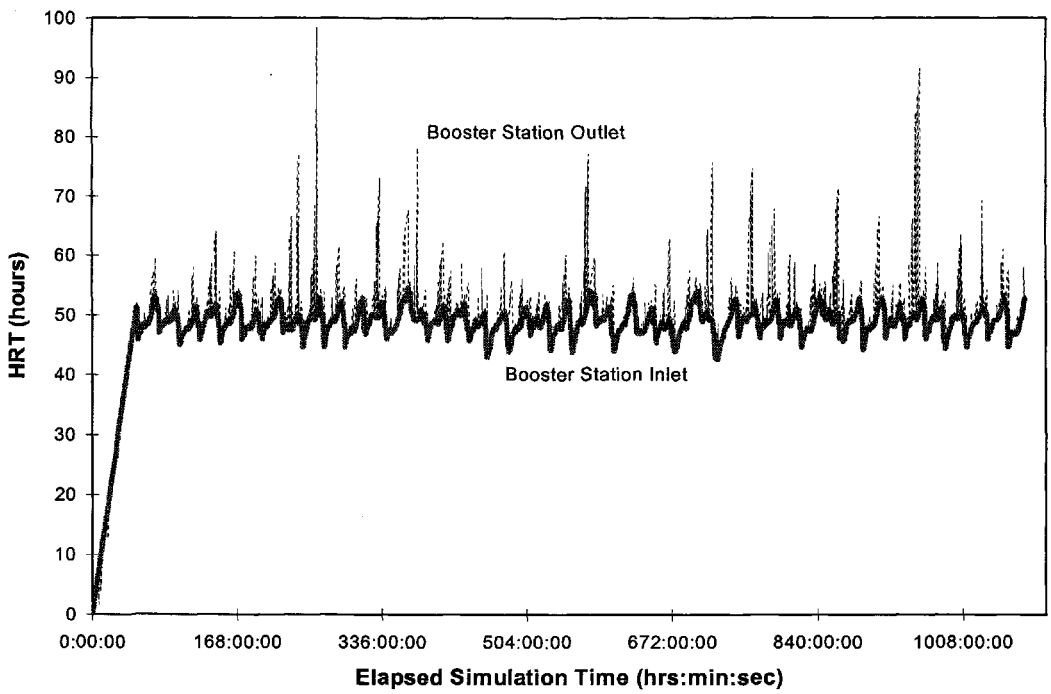

Figure 7.8 Example of water age spikes downstream of the modelled hydro-pneumatic tanks at the Taylorside/Ethelton booster station for the 1st quarter of 2001.

A comparison of the HRT for the monitoring sites is shown in Figure 7.9. Pipeline length, dead ends, the length of the service lines, and low demand cause extended residence times. To illustrate this point, Figure 7.10 includes the site with the longest residence time (Groat farm, refer to Figure 3.1 for location), shown in comparison to the monitoring sites for the same simulation presented in Figure 7.9.

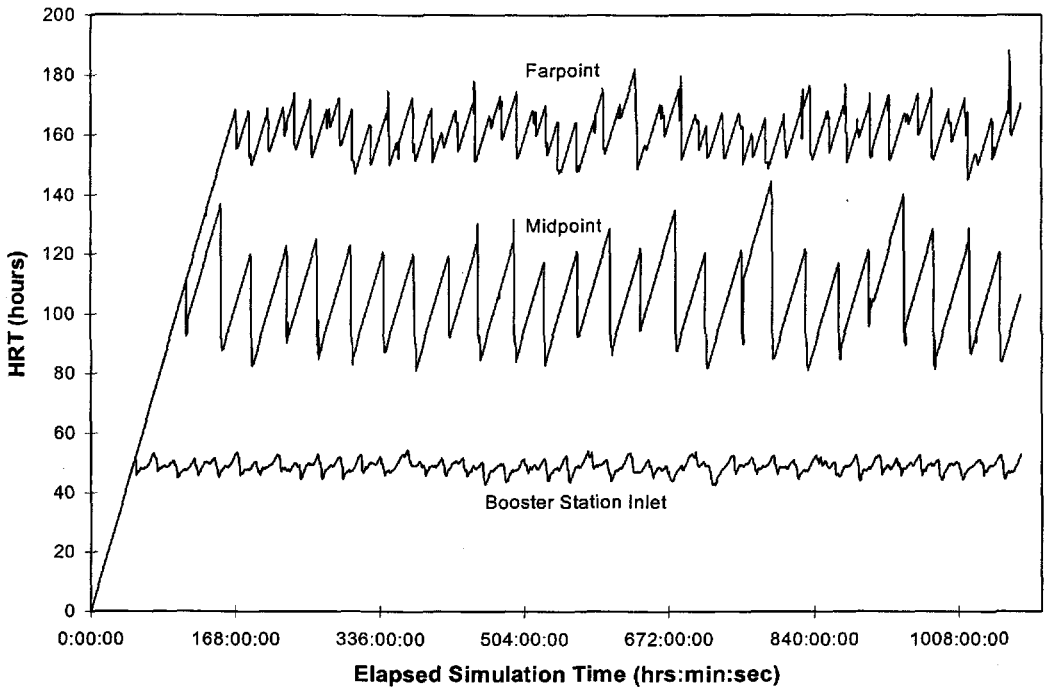

Figure 7.9 Example of modelled HRT for the monitoring sites of the Taylorside/ Ethelton pipeline for the 1st quarter of 2001.

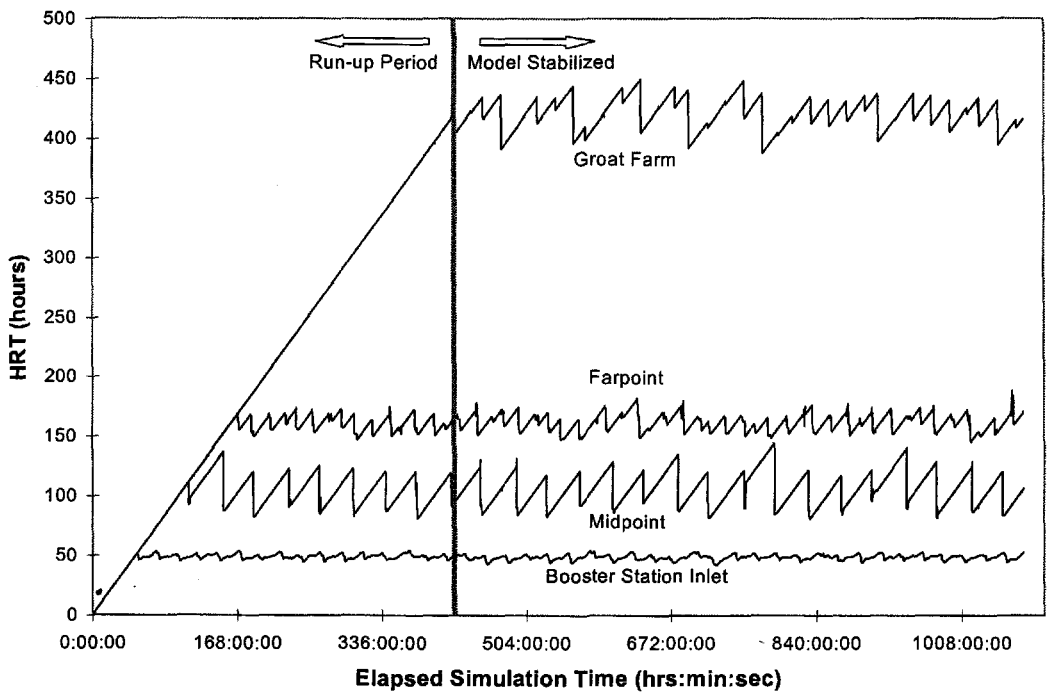

Figure 7.10 Comparison of modelled HRT for the monitoring sites versus that of the Groat farm on the Taylorside/Ethelton pipeline for the 1st quarter of 2001.

EPANET is capable of generating contour plots to illustrate the age of water throughout the distribution system at a given time. This is a useful tool for visualizing the overall system. Figure 7.11 shows an example contour plot of HRTs generated for a specific time within the simulation shown in Figures 7.9 and 7.10. An investigator could use this plot to identify potential problem areas within a network.

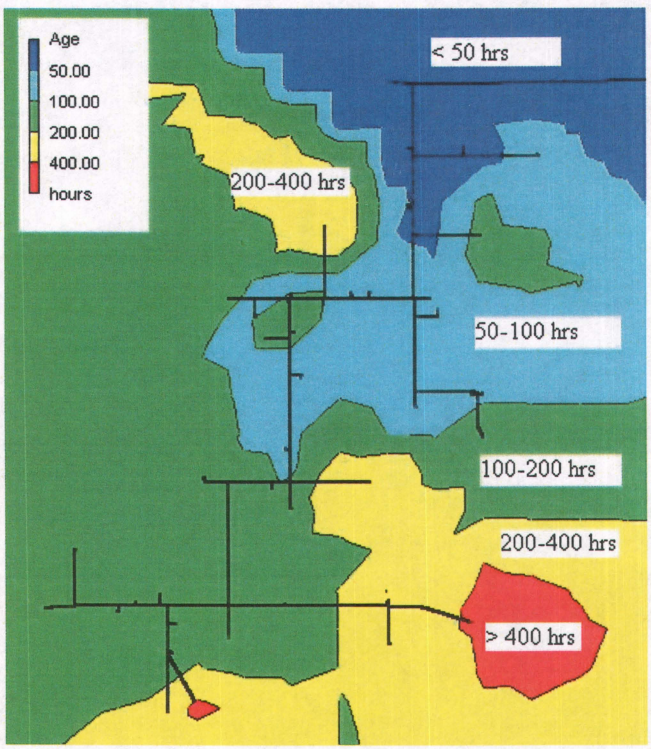

Figure 7.11 Example of HRT contour plot generated within EPANET for the Taylorside/Ethelton pipeline at 846:45 simulation time, 1st Quarter, 2001.

Table 7.3 summarizes the average HRT for each monitoring point and the Groat farm for each of the quarterly simulations. Averages were calculated using the simulation results following the run up period. The length of the run up period was determined for each quarter modelled by examination of the model output. The simulation beyond the run up period was then used to calculate of the average residence times.

Table 7.3 Modelled average hydraulic residence times for the Taylorside/Ethelton pipeline monitoring sites and Groat farm.

Quarter Modelled	Booster Station HRT (hrs)	Midpoint HRT (hrs)	Farpoint HRT (hrs)	Groat Farm HRT (hrs)
3-2000	59	121	224	534
4-2000	60	132	241	585
1-2001	49	107	162	421
2-2001	43	90	134	301
3-2001	51	107	177	315

Peak average residence times occur in the late fall and early winter. Minimum residence times occur in the early spring, when the growing season is beginning and the livestock have not yet been put out to pasture. The residence times for the 3rd quarter of 2001 are significantly lower than those calculated for the same quarter of 2000. This may have been due to the increased demand placed on the system by the dry conditions in 2001. Treated water reservoirs in Saskatchewan are typically designed to store a volume equal to two days of average demand (Saskatchewan Environment, 2002). Thus, the theoretical HRT at average day demand is 48 hours. Since the actual HRT is often less than the theoretical HRT and there is storage within the distribution system, a reasonable estimate of residence time within urban systems would be 24 to 48 hours. Residence times for the Taylorside/Ethelton pipeline are significantly longer than 24 to 48 hours.

The magnitude of variation in the modelled HRTs at each site increased along the length of the pipeline as shown in Figure 7.10. After some thought the reason for this trend becomes apparent. In a system with many users, during a period of elevated demand, the probability is higher that a younger parcel of water will be brought into the vicinity of a model observation site, like the Groat farm for example. This probability is enhanced by the location of high volume users, like those present at the Farpoint, at the extremities of the system.

The maximum residence times, as discussed earlier may be affected by the age spikes originating in the circulating water within the modelled hydro-pneumatic tank. Caution must be exercised when analyzing the maximum residence times as these spikes will have an effect if the aged water is not drawn out of the system by other users in the model before reaching the point being analyzed. A conservative estimate of the maximum residence times would include a correction for the difference in maximum age upstream and downstream of the hydro-pneumatic tank. Table 7.4 shows the maximum estimated residence times for each quarter modelled. This data is provided for illustration purposes and should not be used in analysis as it may not be representative.

Table 7.4 Estimated maximum hydraulic residence times for the Taylorside/Ethelton pipeline monitoring sites and Groat farm.

Quarter Modelled	Booster Station HRT (hrs)	Midpoint HRT (hrs)	Farpoint HRT (hrs)	Groat Farm HRT (hrs)
3-2000	70	148	250	570
4-2000	68	164	269	635
1-2001	55	145	188	450
2-2001	50	116	169	330
3-2001	61	148	207	349

7.1.4 Implications of the Hydraulic Modelling Results

Assuming a typical maximum average residence time of 48 hours in urban systems, the rural system studied showed residence times up to 12 times longer in some cases. The only feasible option for a utility looking to reduce residence time in this situation is an artificial reduction through flushing. Gatel et al. (1998) reported a utility using flushing to reduce residence times in the network and bring higher quality water to the network extremities. The installation of flush points near the ends of a rural network would provide a tool to reduce residence times during critical times of the year. These flush points could be provided by using small (50 mm) hydrants. Wasting water onto the

ground during flushing is not a desirable activity. However, short of reducing the line diameter (which will increase headloss), flushing appears to be the only means available to a rural utility for reducing residence times.

A comparison of modelling by placing a demand at a node upstream of the modelled cisterns (non-intermittent flow in the service line) versus modelling as used for this study (intermittent flow in the service line) was conducted. Both alternatives used daily demand patterns to cause variation in the service line flow rate. The results of this comparison showed the non-intermittent method underestimated the average HRT by approximately 10%. Based on this observation, there is little value in creating an intricate model to estimate average HRT alone. However, if variations in HRT, flow, and pressure are to be studied, as was the case with this research, the effort required to create an intricate hydraulic model is warranted.

7.2 Chlorine Residual Decay Analysis

HRTs generated by the hydraulic model were used in conjunction with the water quality data to establish trends and magnitude of chlorine decay at each of the three Taylorside/Ethelton monitoring sites. The methods of analysis, calculated values and relationships are presented in the following sections.

7.2.1 Method of Analysis for Determination of Decay Coefficients

Daily records for the Melfort water treatment plant contained several free chlorine residual concentrations per day, which were then used to calculate a daily average. The variability of the values taken over the course of the day (in some cases up to 45% of the maximum reading) translated into a variable free chlorine concentrations entering the water treatment plant clear well from one day to the next. Preliminary calculations of chlorine decay, using these daily averages led to highly variable results, and in some cases negative values for the decay coefficient.

To minimize the effect the variable chlorine concentrations had on the decay analysis a cumulative mass curve approach was used. In doing this analysis the assumption was made that the distribution system was incapable of storage between the

water treatment plant outlet and the study site, i.e. volume in equals volume out at any instance. The mass entering the system was determined by multiplying the daily flow volume into the study site (the volume through the Booster Station or entering the user's cistern) by the daily average chlorine concentration measured at the water treatment plant. Similarly the mass exiting the system at the study site was determined by multiplying the flow volume into the study site by the average chlorine concentration measured at the study site. The water treatment plant records from December 14 to December 31 were unavailable so the cumulative curves for 2000 and 2001 were generated separately. Cumulative mass curves for each site are shown in Figures 7.12 through 7.17.

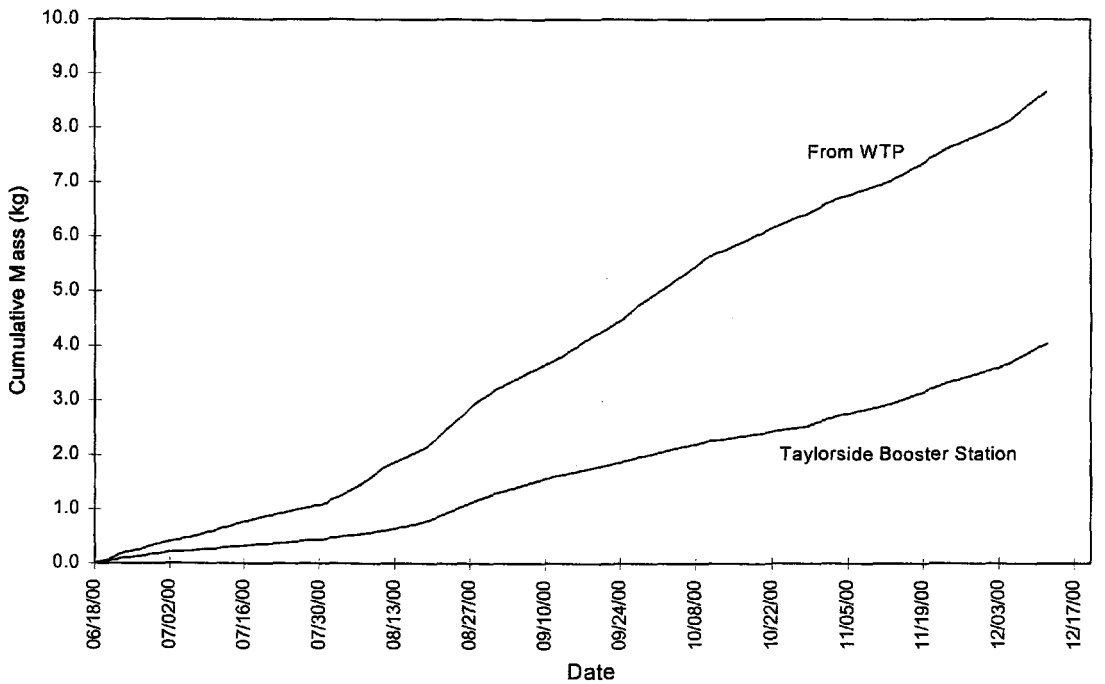

Figure 7.12 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Booster Station monitoring site for data collected in 2000.

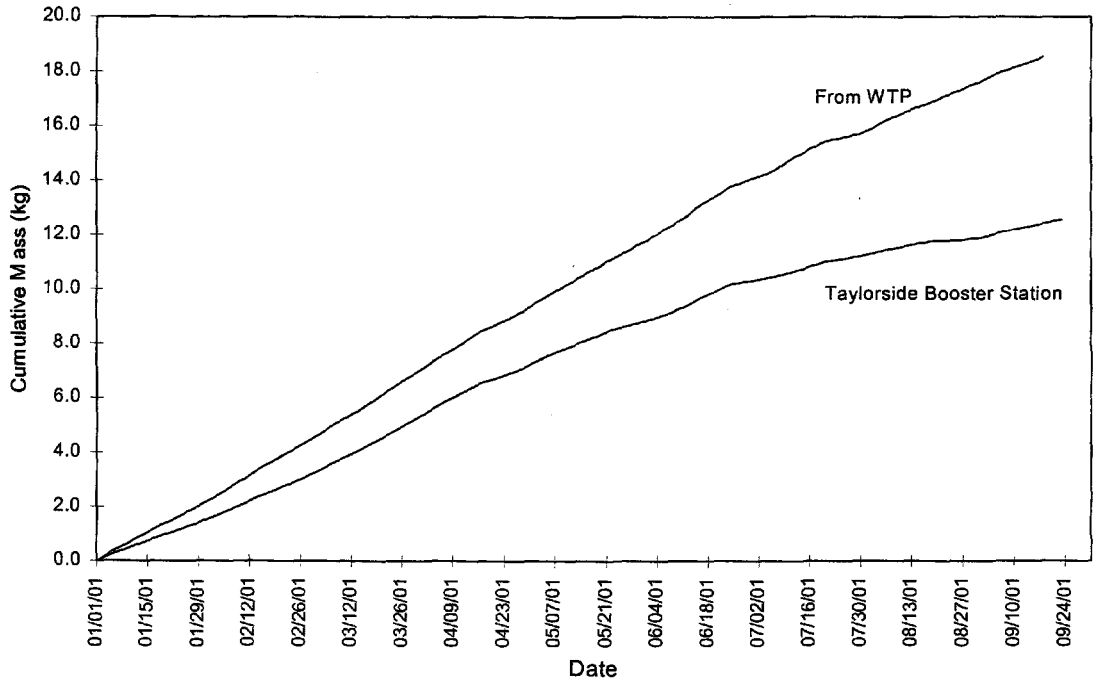

Figure 7.13 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Booster Station monitoring site for data collected in 2001.

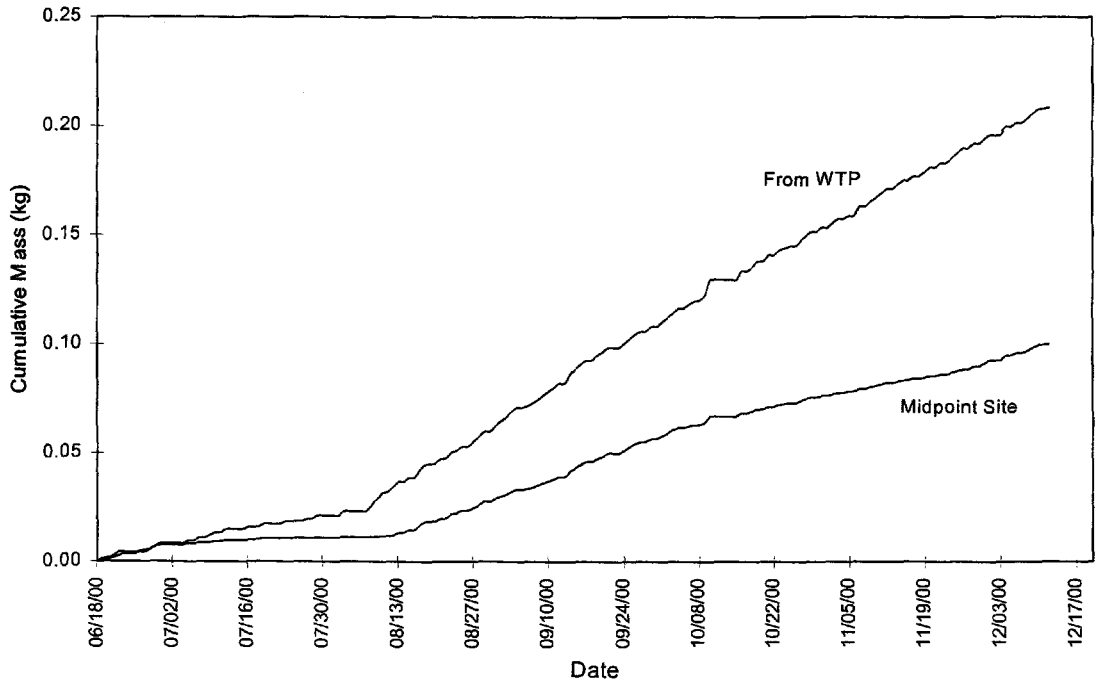

Figure 7.14 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Midpoint monitoring site for data collected in 2000.

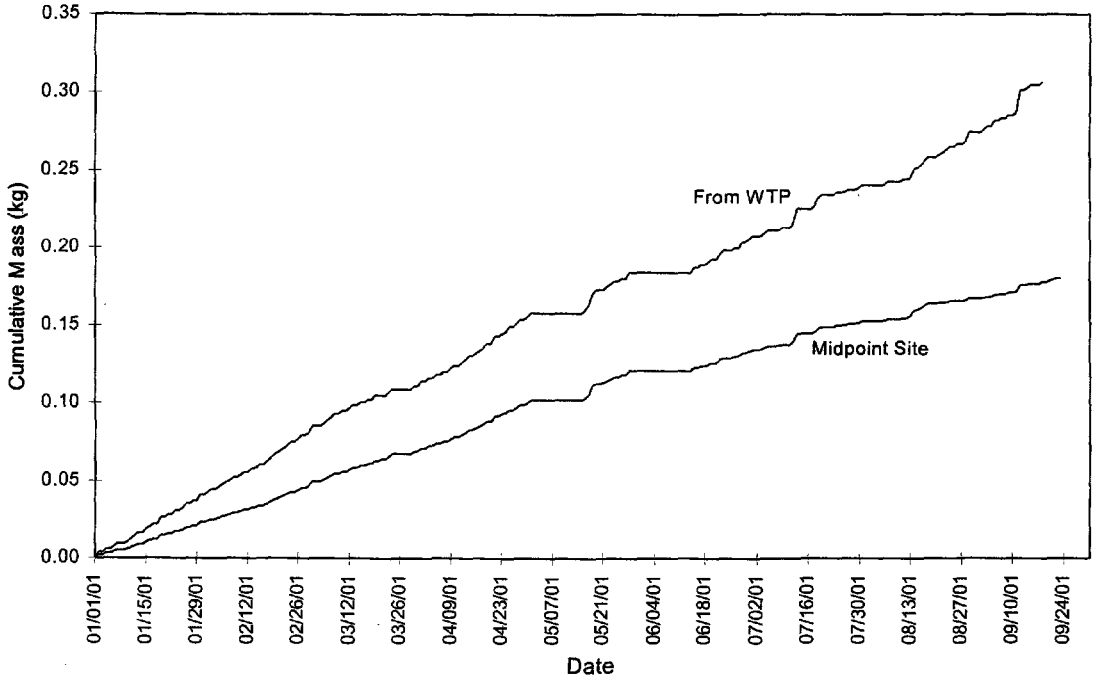

Figure 7.15 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Midpoint monitoring site for data collected in 2001.

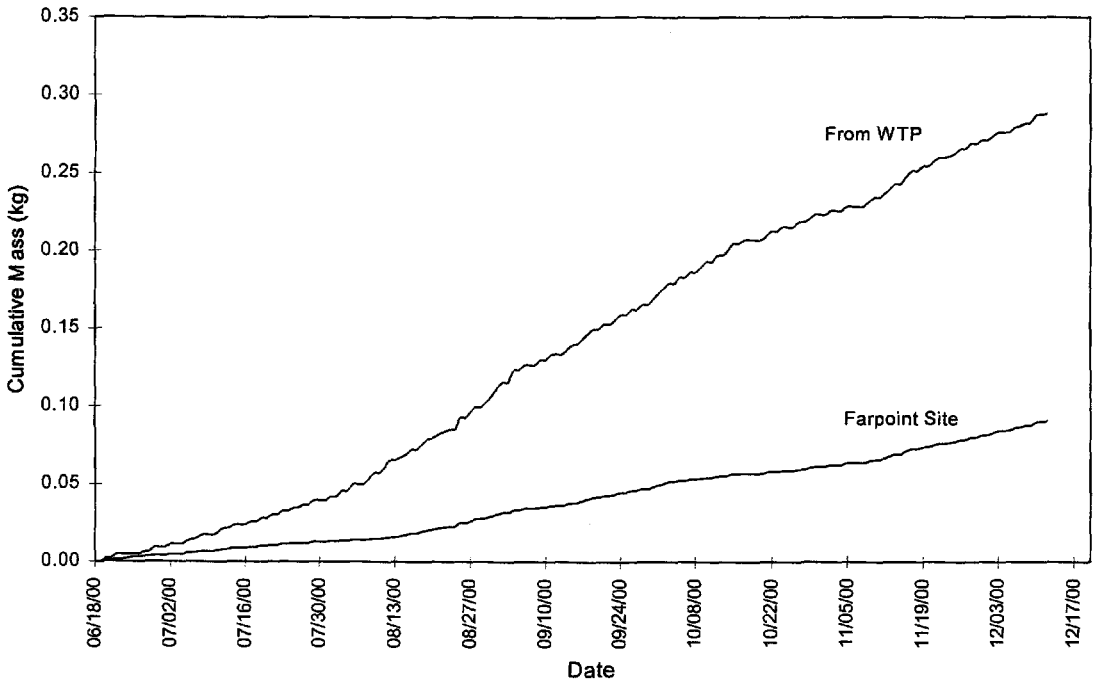

Figure 7.16 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Farpoint monitoring site for data collected in 2000.

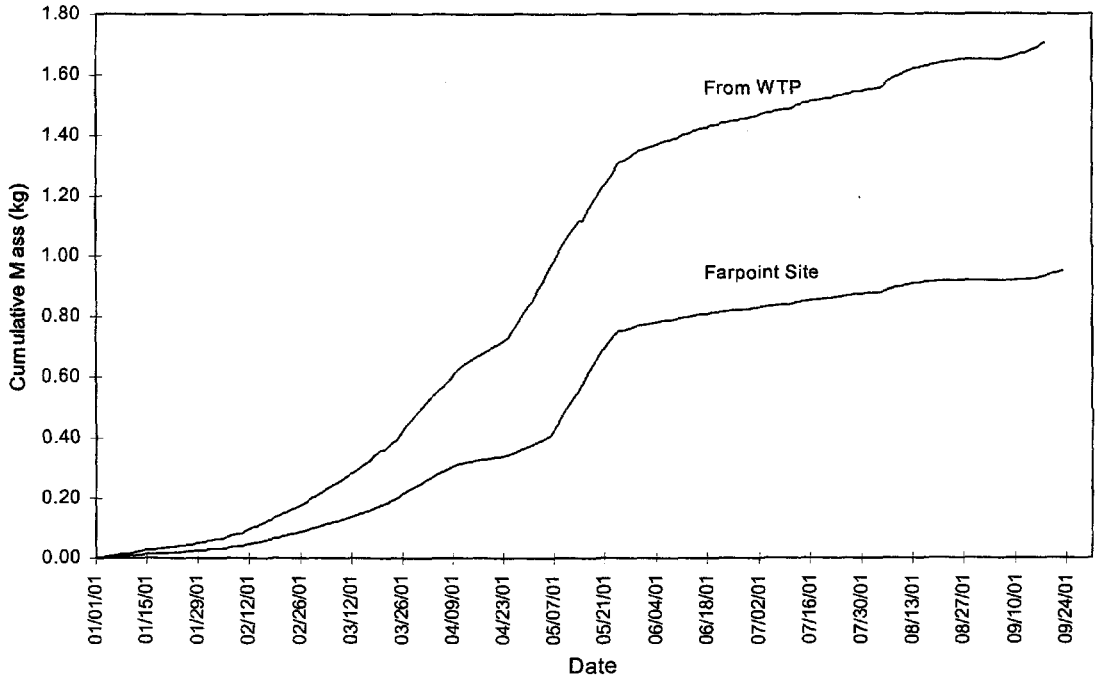

Figure 7.17 Cumulative free chlorine mass curve for the Taylorside/Ethelton pipeline Farpoint monitoring site for data collected in 2001.

Figure 7.18 visually defines the following method of determination of chlorine decay coefficient. Using the mass curves, the day of each site visit was designated as day 'i'. The rate of mass exiting the system at the user site on day 'i' was determined by taking the slope of the output (user site) cumulative mass curve (in kg/d) in the vicinity of day 'i'. The mass exiting the system at the user site on day 'i' was then determined by multiplying the slope by a one day time period, yielding mass M_i. Day 'i' was used to determine the quarter of the year. The typical HRT (determined through EPANET) corresponding to that quarter was then deducted from day 'i'. The slope of the input mass curve in the vicinity of day 'i-HRT' and the mass input for day 'i-HRT' were determined using the same method as for day 'i', yielding mass M_o. Knowing the values of M_i, M_o, and HRT, [2.4] was used to determine the value of rate coefficient 'k'.

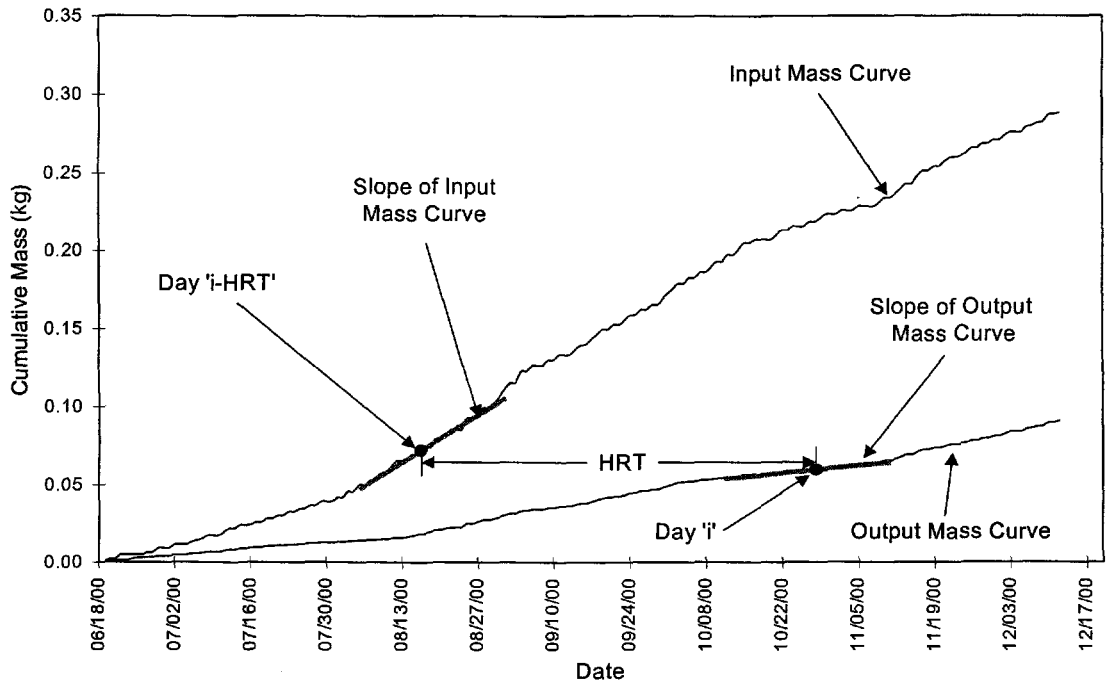

Figure 7.18 Visual definition of terms and method used to determine chlorine decay coefficient.

7.2.2 Decay Coefficients

The calculated values of the decay coefficient over the study period for each site are shown in Figure 7.19. In general, the magnitude of the decay coefficient followed the temperature variation with peak values occurring in the late summer and minimum values in late winter/early spring. Table 7.5 characterizes the decay coefficients for each of the study sites. The decay coefficient was generally larger in magnitude between the water treatment plant and the Taylorside/Ethelton booster station, than between the treatment plant and either of the user sites. This suggests that the major reduction of chlorine residual in the network takes place in the initial portion of the network.

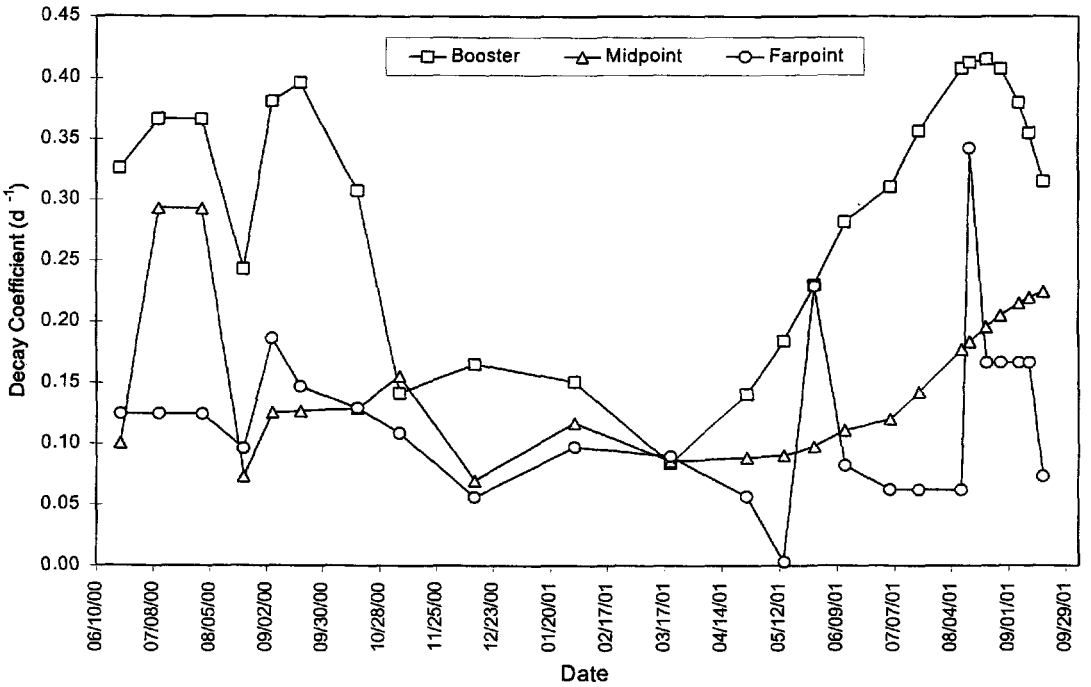

Figure 7.19 Decay coefficients determined for the Taylorside/Ethelton pipeline monitoring sites for the study period.

Table 7.5 Summary of chlorine decay coefficient values (d^{-1}) and variations for the Taylorside/Ethelton pipeline monitoring sites shown in Figure 7.19.

Location	Average (d^{-1})	Maximum (d^{-1})	Minimum (d^{-1})	Average HRT (d)
Taylorside Booster Station	0.297	0.415	0.084	2.18
Midpoint Site	0.152	0.293	0.070	4.64
Farpoint Site	0.122	0.342	0.003	7.82

7.2.3 Relationship of Decay Coefficient with Temperature and DOC

The calculated values of decay coefficient were compared to the temperature and DOC to attempt to determine a relationship between the parameters. The data for each of the three monitoring sites are shown in Figures 7.20 to 7.22. Changes in the decay coefficient followed a pattern very similar to the DOC variation in the early stages of the network (from the water treatment plant to the Taylorside site), but with increasing

residence time the decay patterns appeared to be influenced less by the DOC suggesting that there was another parameter influencing the magnitude of decay.

Rates of reaction will generally increase with temperature. The Arrhenius equation is commonly used to predict the effect of temperature on the rate of reaction. To offset the effect of variable temperature on the calculated decay coefficients, they were corrected to a common temperature of 20° C using the Van't Hoff -Arrhenius equation shown in [7.1].

[7.1] $k_T = k_{20}\theta^{(T-20)}$

where: k_T *is the decay coefficient at temperature T;*
k_{20} *is the decay coefficient at 20 ° C;*
θ *is a constant; and*
T is temperature in ° C.

The EPANET manual (Rossman, 2000) cites Koechling (1998) reporting the value of θ as approximately 1.1 for chlorine. The values of decay coefficient adjusted to 20° C using θ equal to 1.1, are shown plotted together with the DOC for corresponding times (see Figures 7.20 to 7.22). Unfortunately, no mathematical relationship was

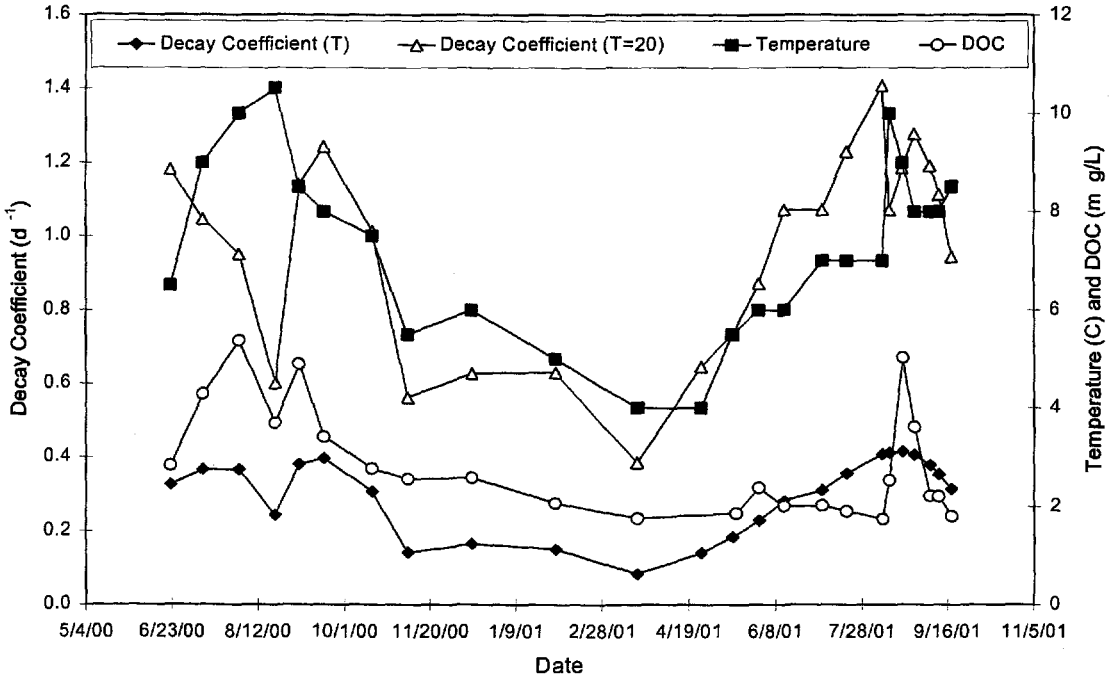

Figure 7.20 Decay coefficient, temperature and DOC values for the Taylorside/Ethelton pipeline Booster Station monitoring site for the study period.

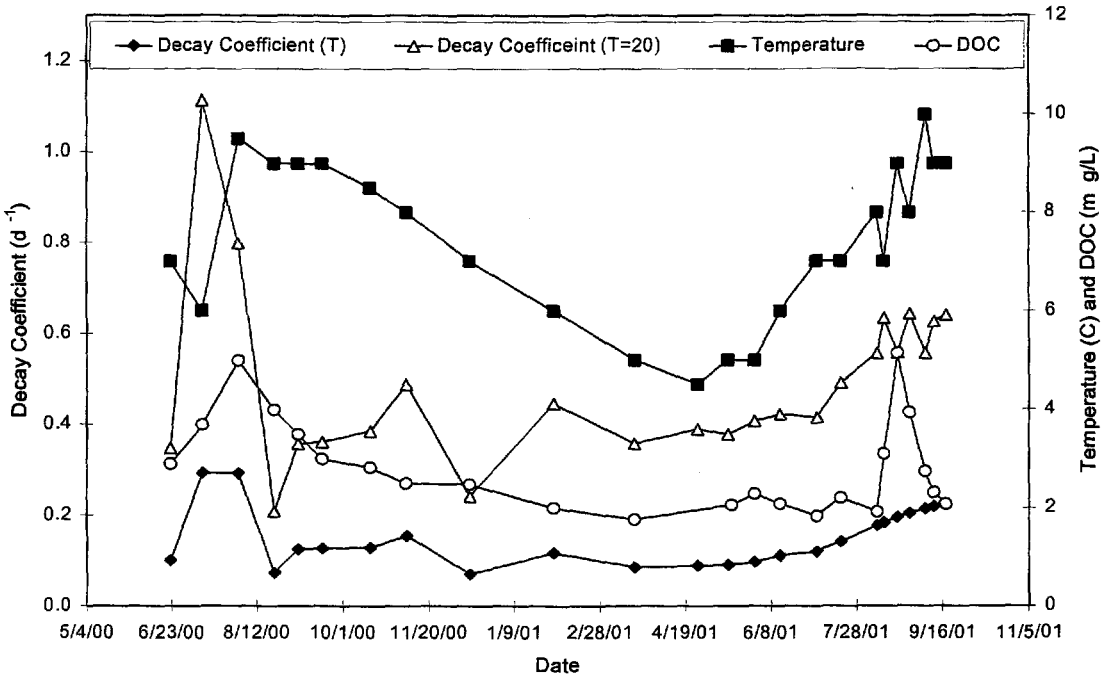

Figure 7.21 Decay coefficient, temperature and DOC values for the Taylorside/Ethelton pipeline Midpoint monitoring site for the study period.

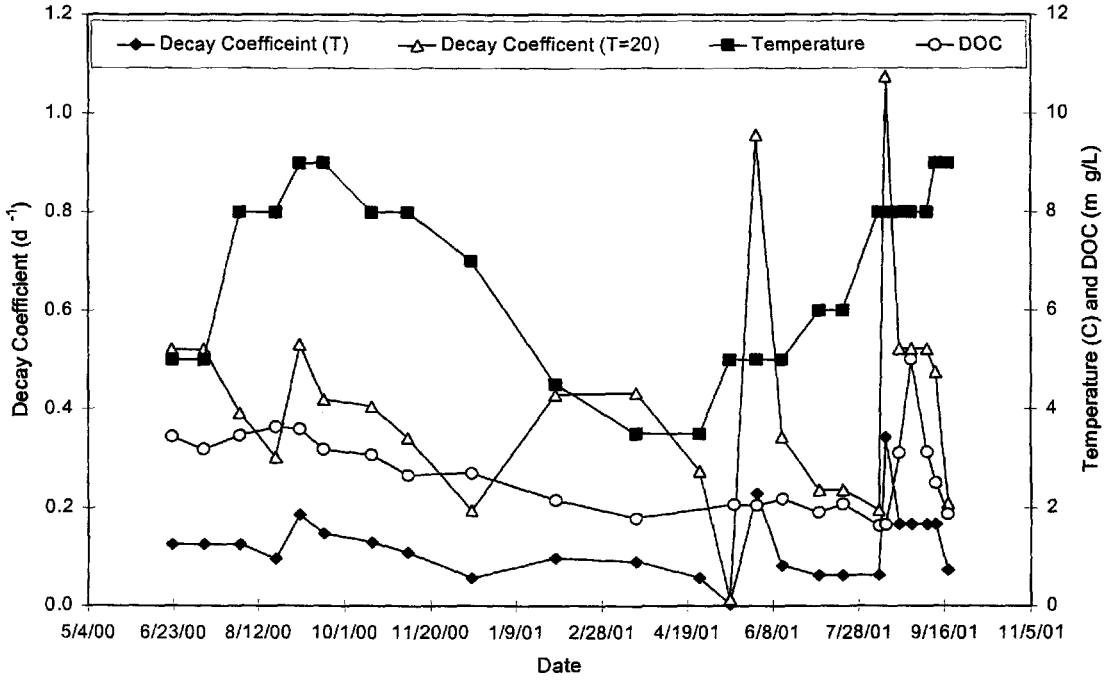

Figure 7.22 Decay coefficient, temperature and DOC values for the Taylorside/Ethelton pipeline Farpoint monitoring site for the study period.

apparent. This is attributed to the fact that the DOC values were instantaneous readings at the end of the reactions that had occurred upstream of the monitoring sites. A determination of the DOC over the course of reaction, which would be expected to correlate with decay coefficient values was not possible due to sampling frequency and variable concentrations of organic matter entering the system.

7.2.4 Interpretation of Chlorine Decay Coefficient Results

The results of the chlorine decay analysis indicated an increase in decay coefficient and decay rate with elevated temperatures and the higher organic carbon levels coinciding with these temperatures. The magnitude of the decay coefficient values showed that the majority of decay occurs in the first part of the distribution system. Table 7.6 summarizes this phenomenon. The values in Table 7.6 were calculated using [2.4] and assuming a free chlorine residual of 1.5 mg/l leaving the water treatment plant with the average values of k and HRT shown in Table 7.6.

Based on these values, nearly half of the free chlorine is depleted in the early stages of transport (i.e. upstream of the booster station). The cause of this phenomenon is believed to be the reaction of easily oxidizable organics with the free chlorine in the early stages of transit, and the more complex organics requiring longer periods to react fully with the free chlorine.

Table 7.6 Calculated chlorine residual depletion using average values of decay coefficient and HRT for the Taylorside/Ethelton pipeline.

Location	C_o (mg/L)	Avg. k (d^{-1})	Avg. HRT (d)	C_i (mg/L)	% Depletion
Booster Station	1.5	0.297	2.18	0.78	48
Midpoint Site	1.5	0.152	4.64	0.74	51
Farpoint Site	1.5	0.122	7.82	0.58	62

The dependency of decay coefficient on the various parameters could not be determined mathematically with any certainty within the scope of this research.

However, some important observations can be noted. The decay of chlorine increased with increasing temperature and higher dissolved organic carbon levels. While temperature is known to be a rate controlling parameter (Van't Hoff – Arrhenius equation), reaction of free chlorine with DOC results in some degree of chlorine decay regardless of temperature. The comparison of temperature corrected decay coefficient and DOC did not yield a viable mathematical relationship. The difference in the decay values between similar levels of organics and temperature could indicate that temperature and DOC are not solely the significant factors controlling the rate and magnitude of chlorine depletion. It is expected that the change in DOC or BDOC from the water plant to the site would more closely correlate with chlorine decay than the absolute values at the end of the reaction. Additionally, the bacterial numbers could be expected to contribute to the decay of chlorine. This intertwined relationship between chlorine decay, organic carbon, bacteria and temperature requires further study and a regression analysis of a large data set to fully appreciate the contribution of each parameter to the decay of residual chlorine.

7.2.5 Comparison of Chlorine Decay Coefficients to Published Results

A direct comparison of decay values between studies is difficult when the contributing factors (temperature, DOC, bacteria, etc.) are not known. An order of magnitude comparison with published values (shown in Table 2.3) is possible. Table 7.7 shows the published results compared to those calculated for the Taylorside/Ethelton pipeline.

It should be noted that the residence times related to the published decay coefficients are only in the order of 12 to 24 hours. These HRTs are much less (50-75%) than those observed in the Taylorside/Ethelton Booster Station. Given a residence time equal to those in this study the published values may have been more similar to those observed in the Taylorside/Ethelton pipeline. Additionally, the published values may have been from distribution systems with metal pipe, which contributes to the depletion of chlorine due to oxidation of the metal.

Table 7.7 Comparison of chlorine decay coefficients.

Decay Coefficient (d^{-1})	Source of Data	Citation
0.1 – 17.7	Urban Distribution System	Vasconcelos et al.(1997)
2.64 – 6.0	Experimental System	Keine et al. (1998)
0.5 – 1.5	Urban Distribution System	Prevost et al. (1998)
0.6 – 0.72	Storage Reservoir Observations	Gagnon et al. (1998)
0.08 – 0.41 [(1)]	Booster Station (Rural Pipeline)	This Study
0.39 – 1.41 [(2)]	Booster Station (Rural Pipeline)	This Study
0.003 – 0.34 [(1)]	User Sites (Rural Pipeline)	This Study
0.01 – 1.11 [(2)]	User Sites (Rural Pipeline)	This Study

[(1)] Average of all monitoring sites at temperatures observed.
[(2)] Average of all monitoring sites, corrected to 20°C.

8.0 KEY OBSERVATIONS, CONCLUSIONS AND RECOMMENDATIONS

The objectives of this study were to document flow and pressure variation in rural pipelines, investigate water quality changes, investigate the formation of biofilms, investigate chlorine decay coefficient, and develop a computer model to estimate the residence times.

8.1 Key Observations and Conclusions

8.1.1 Flow and Pressure Variation

The demands placed on a rural water distribution network vary depending on the season. In contrast to demand patterns in urban networks, the peak demand on rural systems occurred during the winter when livestock watering was required. Peaks also occurred in spring when the growing season began and livestock had not yet been put out to pasture. Another peak occurred in the late summer during the period when lawn and garden watering are typically occurring. Minimum demand occurred in late fall and early winter when agricultural activity was at its lowest. The effects of low precipitation may have caused the increased system demand observed in the summer of 2001.

Pressures were found to vary significantly at the extremities of rural water networks occasionally causing unacceptably low pressures at these sites. An increase in pressure, in the absence of flow control, appeared to do little more than increase flow rates into the users nearest the booster station. The headloss as a result of filling cisterns at the system periphery was notable due to the considerable length of small diameter line leading to these locations. The loss of system pressure during these events could be identified elsewhere in the network.

The rural maximum daily peak flow factor identified in this study was lower than published typical maximum daily peak factors for urban systems which do not employ cisterns for demand equalization. Rural maximum hourly peak factors are similar to published results for urban systems. Specifically, for the Taylorside/Ethelton rural water pipeline, the average maximum hourly peak factor (max. hour/avg. day) was 4.53 and the average maximum daily peak factor (max. day/avg. day) was 1.97. The low pressure low flow design philosophy used to design rural water distribution networks appears result in lower maximum daily peak factors but does little for maximum hourly values with the absence of flow control at the user sites. The daily flow patterns in the Taylorside/ Ethelton pipeline do not follow the diurnal patterns present in urban systems.

Based on the observed demand and pressure fluctuations it appears the booster pump is operating at or near its maximum capacity during periods of high demand. Installation of flow regulators at the users near the pumphouse may improve system pressure residuals at the extents of the network through lower flow rates (and thus less headloss) in the upstream end of the network.

8.1.2 Distribution System Water Quality

The results of the water quality monitoring suggested that the typically low temperatures, believed to be a function of subsurface ground temperature and long HRTs, mitigate the deterioration of water quality in the two pipelines monitored. Seasonal peak values of organic matter, bacterial populations, and chlorine decay were found to coincide with peak temperature periods, typically occurring in late August and early September. Chlorine residuals were noted to reach minimum values during these periods. A change to poorer quality source water was noted to adversely affect distribution system water quality. This was clearly demonstrated on the Taylorside/ Ethelton pipeline when a switch to a poor quality alternate raw water supply introduced high levels of DOC into the network, causing accelerated chlorine decay and promoting growth of bacterial populations within the network.

Low temperatures during the winter months slowed the chemical and biological reactions between the water quality parameters studied and promoted stability of these values within the two rural water pipelines. Temperatures reached minimum values in late March and early April, and coincided with the lowest variability of organic concentrations and the highest chlorine residuals in the Taylorside/Ethelton pipeline.

Variable residual chlorine concentrations fed to the Taylorside/Ethelton pipeline are suspected to be a contributing factor to the erratic growth and decline of bacterial populations and changes in the concentrations of bacterial substrate. However, it was only when the free chlorine was completely depleted within the network that viable organisms were detected. Even then, the concentrations were well below the threshold values above which adverse health effects are believed to occur. With the exception of small portions of the peak temperature periods, the free chlorine residuals remained above the threshold value of 0.2 mg/l required to inhibit biofilm formation.

On occasion, particularly during the summer months, BDOC concentrations in the networks exceeded the biofilm threshold value of 0.2 mg/l. Limited growth was noted on these occasions but development was believed to be largely mitigated by the temperature of the water. In the early stages of the Coteau Hills regional network the temperature exceeds the threshold value of 15 °C, but despite this, temperatures in the Lucky Lake North branch did not exceed 14 °C for the duration of the study. Temperatures of the water leaving the Melfort water treatment plant peaked at 14 °C but temperatures within the Taylorside/Ethelton network did not exceed 11°C.

In general, temperatures and chlorine residuals were observed to decrease with distance from the head of the networks, implying a decrease with residence time. No such trend could be clearly identified for DOC, BDOC, or bacteria counts. During the peak temperature periods a general trend of the parameters is difficult to determine due to variability of the values caused by accelerated interaction. The frequency of sampling also may have been insufficient to accurately record the fate of distinct parcels of water as they moved through the system during these periods. Higher resolution sampling was not possible within the financial constraints of this study.

Poor water quality appears to be a seasonal problem in rural water distribution networks occurring in late summer during the period of study. Breaches in the threshold values of BDOC and residual free chlorine believed to mitigate biofilm control were observed. Although temperature rose above the threshold values in the regional lines feeding the two rural water distribution networks studied, temperature remained below the threshold value in both of the branch networks. The low temperatures are believed to have prevented significant regrowth within the two branch networks.

8.1.3 Biofilm Analysis

Pipe samples from each of the networks were analyzed for biofilm density. The measured biofilm densities were very low compared to some published values. The viable biofilm densities from the treated water distribution network were two orders of magnitude less than those found in the untreated water distribution network samples. These results were found in the presence of similar levels of DOC at the user sites, with temperatures approximately 4 °C cooler in the treated water distribution network. Low temperatures, maintenance of a chlorine residual (in the case of the Taylorside/Ethelton pipeline) and possibly substrate limitation are believed to inhibit biofilm formation in these networks.

8.1.4 Hydraulic Modeling

Modelling showed residence times peak in the late fall and early winter and reach minimum values in the early spring. This general trend could also be concluded by observation of the average system demands. The HRTs for the Taylorside/Ethelton monitoring sites were found to vary from 43 to 60 hours at the Booster Station, from 90 to 132 hours at the Midpoint, and 134 to 241 hours at the Farpoint. The maximum modelled average residence times were observed at the Groat farm which ranged from 381 to 585 hours. These times are substantially longer than those in urban water distribution networks. Extended residence times can cause excessive deterioration of water quality due to the period allowed for reaction of the parameters. The one benefit of long hydraulic residence time is the reduction of water temperatures which, as

discussed above, has a mitigating influence on water quality deterioration. Water quality issues during extended residence times are often seasonal owing to the increase in activity at elevated temperatures. Flushing of the network to reduce the residence time is an acceptable solution to the seasonal problems.

8.1.5 Chlorine Decay Analysis

The chlorine decay analysis showed, on average, nearly 50 % of the initial free chlorine was consumed before it reached the Taylorside/Ethelton pipeline (2.2 days average HRT). On average, 51% and 62% of the initial free chlorine was depleted by the Midpoint (4.6 days avg. HRT) and Farpoint (7.8 days avg. HRT), respectively. The majority of reaction occurs in the early stages of transit. The decay coefficients determined in this study are similar in magnitude, but less than, the values reported in literature.

A single mathematical relationship for decay coefficient as a function of both temperature and DOC could not be established. The relationship might become apparent for the early stages of the network if the DOC was measured over the course of reaction. The latter stages of transit show higher variability in the decay coefficient values for similar temperatures and DOC concentrations. This suggests that as distance from the source increases, temperature and DOC, have a decreased effect on decay coefficient which becomes more dependent on some other parameter not included in the chlorine decay analysis.

8.2 Recommendations

8.2.1 Operational and Monitoring Recommendations

Rural water pipelines should be routinely sampled for chlorine residuals, temperature and bacteria counts. The frequency of this sampling should be increased during the periods when water quality is most susceptible to degradation. The most susceptible periods were in spring (possibly due to runoff or reservoir turnover in raw water sources), and in the late summer when temperatures and reaction rates reach peak

values. Tracking of the pipeline water temperature should aid in identifying periods of susceptibility as they approach.

Operators of branch networks should be notified of changes in water source or drastic variations in water quality at the source so that they may increase the frequency of their monitoring program and be prepared to take appropriate measures before water quality deteriorates to unacceptable levels.

Flushing points should be installed on networks with high residence times to temporarily improve water quality throughout the system during the periods when water quality degradation is at its highest. In order to avoid wasting water onto the ground, livestock producers might be notified of the flushing operation and, if interested, be allowed to collect the poor quality water for their livestock, which are less susceptible to the health effects of concern to human consumers.

Local flush points should be provided at consumer connections to allow for reductions in residence time in lengthy services. These flush points could be used in the collection of samples for determination of quality at the location.

During all flushing operations, samples should be taken and analyzed for free chlorine, turbidity, HPC and coliform bacteria to indicate the degree of or potential for bacterial proliferation within that leg of the network.

Sections of pipe recovered from the system during maintenance activity or repair should periodically be preserved and analyzed for biofilm formation.

Pumping systems and accessible portions of the network downstream should be equipped with flow and pressure monitoring devices capable of recording or reporting daily averages as a minimum but preferably hourly or sub-hourly values. The data, once collected, should be analyzed for the demand patterns and any operational anomalies.

Flow limitation in the vicinity of booster stations and other high pressure areas should be considered to limit the pressure loss in other locations of the network. Hydraulic modelling to examine the effects of limitation would be beneficial.

Hydraulic computer models with software such as EPANET are a powerful yet underutilized tool in the identification of potential problem areas in networks. Once

constructed, the computer model can be used to estimate the effects of additional users or changes in the pumping regime, operational procedures or pressure and flow control equipment location.

Designers should use residence time analyses and decay calculations (using maximum decay coefficient values) to assess proposed expansions or construction of additional lateral networks in the area. While the decay coefficients identified in this study only directly apply to the Taylorside/Ethelton network, they may be a useful tool in other locations. Applying these values during the design of new rural networks or expansions may identify the need for booster chlorination facilities or flush points to maintain an adequate residual at the extents of the networks. Without a mathematical representation of the relative influence of the parameters causing decay in the study network, the decay values cannot be applied directly to networks receiving water from a different source. However, they may provide a basis for determining preliminary values for disinfection dosage equipment or locating booster chlorination facilities. The investigator must have an understanding of the general interaction between parameters influencing decay and adjust the decay coefficient values accordingly for design.

8.2.2 Recommendations for Future Study

The quality of water in the storage cisterns at the monitoring sites was not investigated. There is potential for depletion of chlorine and growth of organisms within these storage tanks. The potential for change in cistern water quality between solenoid actuations merits further investigation and the effects of decreased working volumes in the cisterns could be included as part of the study.

The changes in source water at the Taylorside/Ethelton pipeline presented a possible topic of research. The distribution of dissolved organic carbon fractions in different water sources warrants further study. The investigation would determine the chemical make up and concentration of easily assimilable organics utilized by bacteria in the network for re-growth. The difference in the composition of dissolved organic carbon from different sources such as ground waters, stagnant shallow impoundments, and flowing waters would provide an understanding of the chemical make up of

Saskatchewan water sources. Characterization of these fractions may lead to an improvement in the methods of removal employed in water treatment processes and a shift towards the production of biologically stable drinking water.

The current study showed that the formation of biofilms is largely inhibited by low temperatures present in the network. Investigation of a system where temperatures would be less dominant in the control of biofilm formation is warranted as the two systems studied may not be representative of conditions prevailing elsewhere in Saskatchewan and Canada.

Using ground temperature to cool treated water prior to delivery as a control for water quality deterioration is a topic that warrants further investigation.

Future sampling of pipes for biofilms should include the high flow areas at the head of the systems as they may be more likely to have organic carbon concentrations adequate for biofilm growth, even in the presence of higher chlorine residuals. If the pipes cannot be sampled directly, and valve or meter vaults are available, monitoring devices can be installed at these locations. These devices take a split stream off of the main and monitor biofilm formation on removable coupons or biofilm formation potential monitors.

Future studies requiring determination of the fraction of organics promoting growth, such as BDOC, should use an alternate method of determination from the one used herein. The lengthy incubation times and variable results suggest that this method may not be best suited for the types of water present in the networks studied. The low concentrations of BDOC measured also suggest that one of the methods of AOC determination, while more labour intensive, would more accurately quantify the concentration of organic matter available to bacteria.

If decay analyses are to be completed, the changes in organic carbon concentrations due to reaction from source to user should be monitored as well as the initial and final values of free chlorine. In this way a relationship between the magnitude of decay and the organic concentration may be developed.

Future sampling assemblies should include new and separate pressure and flow control devices to ensure consistency of sampling rates between sites.

The EPANET model appeared to underestimate the volumes observed in the study. The observed values were generated from high resolution observations and averaged to an interval of 15 minutes. Due to a limited computing power, the model resolution was set to 15 minute time steps, which resulted in instantaneous flow rates at 15 minute intervals. A topic for future research may be to investigate the effect of increasing model resolution on the error between modelled and observed volumes.

If distribution system modelling is employed for determining the average value of HRT only, use of constant demands at the user nodes should provide a reasonable estimate of the average HRT. If other parameters are to be investigated, for example variation in flow and pressure, a detailed model is required.

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APPENDIX A: HYDRAULIC DATA FILENAME LISTING

The attached data CD contains the flow and pressure values collected from the monitoring sites over the study period. The data for the user sites was collected with a "day of year" notation. The file named 'Day Of Year.xls' shows the day of year for each date in 2000 and 2001. The filename listing for the hydraulic data is shown below.

Hydraulic Data

- Day Of Year.xls
- Lucky Lake North Pipeline Hydraulic Data
  - 1 LLN Booster Station HD
    - LLN Feed HD (daily average) for Study Period.xls
  - 2 LLN Midpoint HD
    - LLN Midpoint HD 2000
      - LLN Midpoint HD day 167 - day 179 2000.xls
      - LLN Midpoint HD day 180 - day 208 2000.xls
      - LLN Midpoint HD day 209 - day 221 2000.xls
      - LLN Midpoint HD day 222 - day 241 2000.xls
      - LLN Midpoint HD day 242 - day 255 2000.xls
      - LLN Midpoint HD day 256 - day 271 2000.xls
      - LLN Midpoint HD day 272 - day 283 2000.xls
      - LLN Midpoint HD day 284 - day 304 2000.xls
      - LLN Midpoint HD day 305 - day 329 2000.xls
      - LLN Midpoint HD day 330 - day 355 2000.xls
      - LLN Midpoint HD day 356 - day 366 2000.xls
    - LLN Midpoint HD 2001
      - LLN Midpoint HD day 1 - day 32 2001.xls
      - LLN Midpoint HD day 33 - day 79 2001.xls

LLN Midpoint HD day 80 - day 114 2001.xls

LLN Midpoint HD day 115 - day 134 2001.xls

LLN Midpoint HD day 135 - day 149 2001.xls

LLN Midpoint HD day 150 - day 165 2001.xls

LLN Midpoint HD day 166 - day 186 2001.xls

LLN Midpoint HD day 187 - day 203 2001.xls

LLN Midpoint HD day 204 - day 219 2001.xls

LLN Midpoint HD day 220 - day 226 2001.xls

LLN Midpoint HD day 227 - day 231 2001.xls

LLN Midpoint HD day 232 - day 238 2001.xls

LLN Midpoint HD day 239 - day 252 2001.xls

LLN Midpoint HD day 253 - day 274 2001.xls

3 LLN Farpoint HD

LLN Farpoint HD 2000

LLN Farpoint HD day 169 - day 179 2000.xls

LLN Farpoint HD day 189 - day 199 2000.xls

LLN Farpoint HD day 200 - day 208 2000.xls

LLN Farpoint HD day 211 - day 221 2000.xls

LLN Farpoint HD day 222 - day 241 2000.xls

LLN Farpoint HD day 242 - day 255 2000.xls

LLN Farpoint HD day 256 - day 271 2000.xls

LLN Farpoint HD day 272 - day 283 2000.xls

LLN Farpoint HD day 284 - day 304 2000.xls

LLN Farpoint HD day 305 - day 325 2000.xls

LLN Farpoint HD day 326 - day 355 2000.xls

LLN Farpoint HD day 356 - day 366 2000.xls

LLN Farpoint HD 2001

LLN Farpoint HD day 1 - day 32 2001.xls

LLN Farpoint HD day 33 - day 79 2001.xls

LLN Farpoint HD day 80 - day 114 2001.xls

LLN Farpoint HD day 115 - day 134 2001.xls

LLN Farpoint HD day 135 - day 149 2001.xls

LLN Farpoint HD day 150 - day 165 2001.xls

LLN Farpoint HD day 166 - day 186 2001.xls

LLN Farpoint HD day 187 - day 203 2001.xls

LLN Farpoint HD day 204 - day 219 2001.xls

LLN Farpoint HD day 220 - day 226 2001.xls

LLN Farpoint HD day 227 - day 231 2001.xls

LLN Farpoint HD day 232 - day 238 2001.xls

LLN Farpoint HD day 239 - day 252 2001.xls

LLN Farpoint HD day 253 - day 274 2001.xls

Taylorside Ethelton Pipeline Hydraulic Data

1 TE Booster Station HD

TE Booster Station HD for Study Period.xls

2 TE Midpoint HD

TE Midpoint HD 2000

TE Midpoint HD day 165 - day 173 2000.xls

TE Midpoint HD day 174 - day 193 2000.xls

TE Midpoint HD day 194 - day 213 2000.xls

TE Midpoint HD day 214 - day 234 2000.xls

TE Midpoint HD day 235 - day 248 2000.xls

TE Midpoint HD day 249 - day 262 2000.xls

TE Midpoint HD day 263 - day 291 2000.xls

TE Midpoint HD day 292 - day 311 2000.xls

TE Midpoint HD day 312 - day 348 2000.xls

TE Midpoint HD day 349 - day 366 2000.xls

TE Midpoint HD 2001

TE Midpoint HD day 1 - day 31 2001.xls

TE Midpoint HD day 32 - day 78 2001.xls

TE Midpoint HD day 79 - day 115 2001.xls

TE Midpoint HD day 116 - day 164 2001.xls

TE Midpoint HD day 165 - day 185 2001.xls

TE Midpoint HD day 186 - day 199 2001.xls

TE Midpoint HD day 200 - day 220 2001.xls

TE Midpoint HD day 221 - day 224 2001.xls

TE Midpoint HD day 225 - day 232 2001.xls

TE Midpoint HD day 233 - day 239 2001.xls

TE Midpoint HD day 240 - day 248 2001.xls

TE Midpoint HD day 249 - day 253 2001.xls

TE Midpoint HD day 254 - day 276 2001.xls

3 TE Farpoint HD

TE Farpoint 2000

TE Farpoint HD day 165 - day 173 2000.xls

TE Farpoint HD day 174 - day 192 2000.xls

TE Farpoint HD day 193 - day 213 2000.xls

TE Farpoint HD day 214 - day 234 2000.xls

TE Farpoint HD day 235 - day 248 2000.xls

TE Farpoint HD day 249 - day 290 2000.xls

TE Farpoint HD day 291 - day 311 2000.xls

TE Farpoint HD day 312 - day 348 2000.xls

TE Farpoint HD day 349 - day 366 2000.xls

TE Farpoint 2001

TE Farpoint HD day 1 - day 31 2001.xls

TE Farpoint HD day 32 - day 78 2001.xls

TE Farpoint HD day 79 - day 115 2001.xls

TE Farpoint HD day 116 - day 148 2001.xls

TE Farpoint HD day 149 - day 163 2001.xls

TE Farpoint HD day 164 - day 185 2001.xls

TE Farpoint HD day 186 - day 199 2001.xls

TE Farpoint HD day 200 - day 220 2001.xls

TE Farpoint HD day 221 - day 224 2001.xls

TE Farpoint HD day 225 - day 232 2001.xls

TE Farpoint HD day 233 - day 276 2001.xls

APPENDIX B: WATER QUALITY DATA FILENAME LISTING

The attached data CD contains values of water quality parameters collected from the monitoring sites over the study period. The filename listing for the water quality data is shown below.

Water Quality Data

- Lucky Lake North Pipeline Water Quality Data
  - LLN DOC and BDOC.xls
  - LLN Epifluorescent Bacteria Counts.xls
  - LLN Particle Counts.xls
  - LLN Temperature.xls
  - LLN Turbidity.xls
- Taylorside Ethelton Pipeline Water Quality Data
  - TE Chlorine Residuals.xls
  - TE DOC and BDOC.xls
  - TE Epifluorescent Bacteria Counts.xls
  - TE HPC.xls
  - TE Particle Counts.xls
  - TE Temperature.xls
  - TE Turbidity.xls

APPENDIX C: EPANET MODEL SIMULATION AND CHLORINE DECAY COEFFICIENT DATA FILENAME LISTING

The attached data CD contains quarterly model simulations and chlorine decay coefficient results for the period of study. EPANET files, output from the quarterly simulations for nodes of interest, cumulative mass curves and decay coefficient values are included. The filename listing for the data is shown below.

Model Data

- Decay Coefficient Data
  - Decay Coefficient.xls
  - Booster Station Cumulative Mass Curve Data.xls
  - Midpoint Cumulative Mass Curve Data.xls
  - Farpoint Cumulative Mass Curve Data.xls
- EPANET files
  - TE HRT qtr 3 2000.net
  - TE HRT qtr 4 2000.net
  - TE HRT qtr 1 2001.net
  - TE HRT qtr 2 2001.net
  - TE HRT qtr 3 2001.net
- Model Simulation Results
  - TE qtr 3 -2000 Model Simulation results.xls
  - TE qtr 4 -2000 Model Simulation results.xls
  - TE qtr 1 -2001 Model Simulation results.xls
  - TE qtr 2 -2001 Model Simulation results.xls
  - TE qtr 3 -2001 Model Simulation results.xls